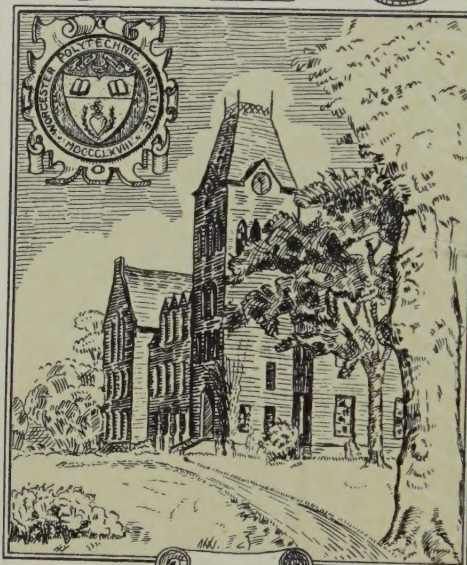
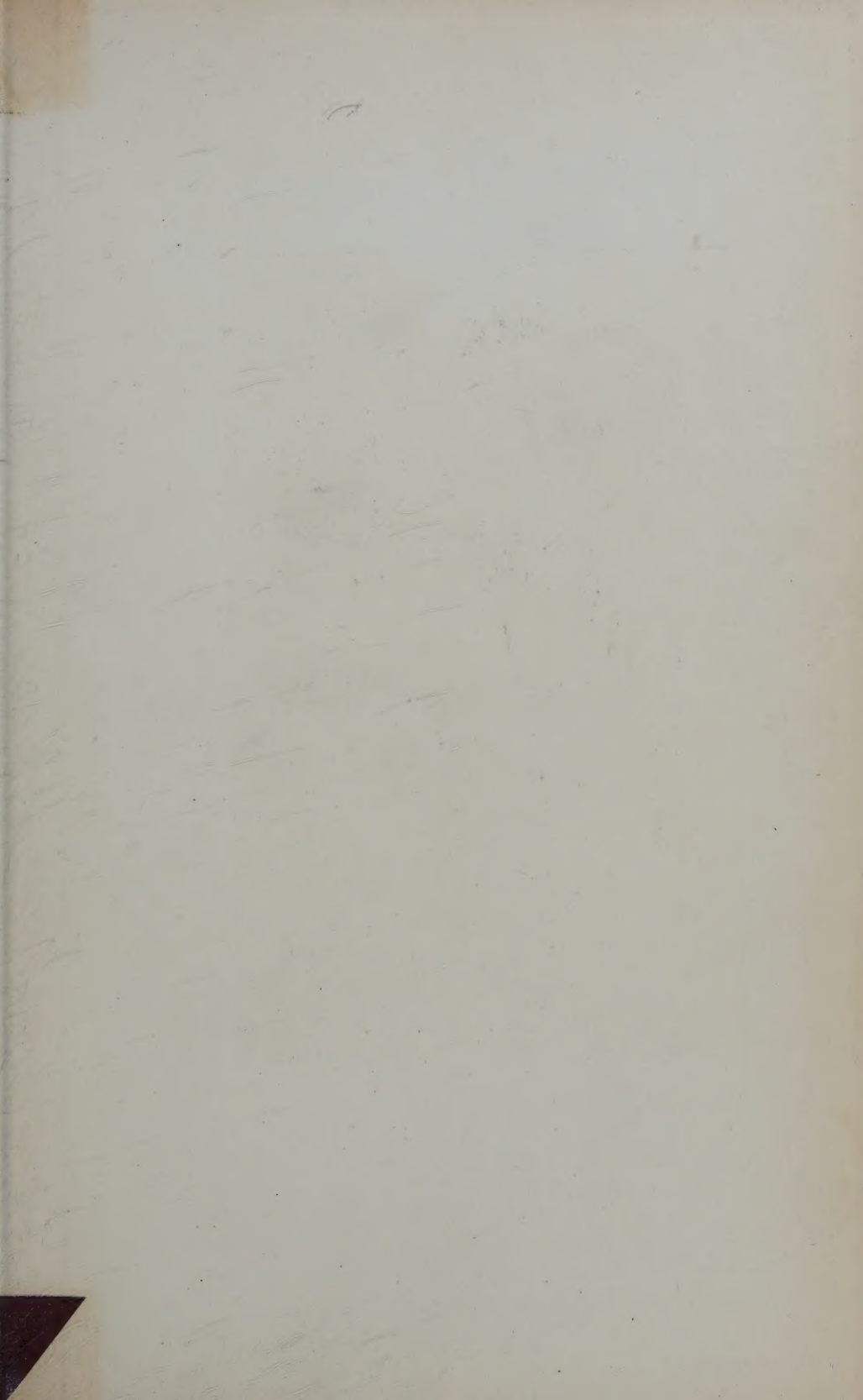
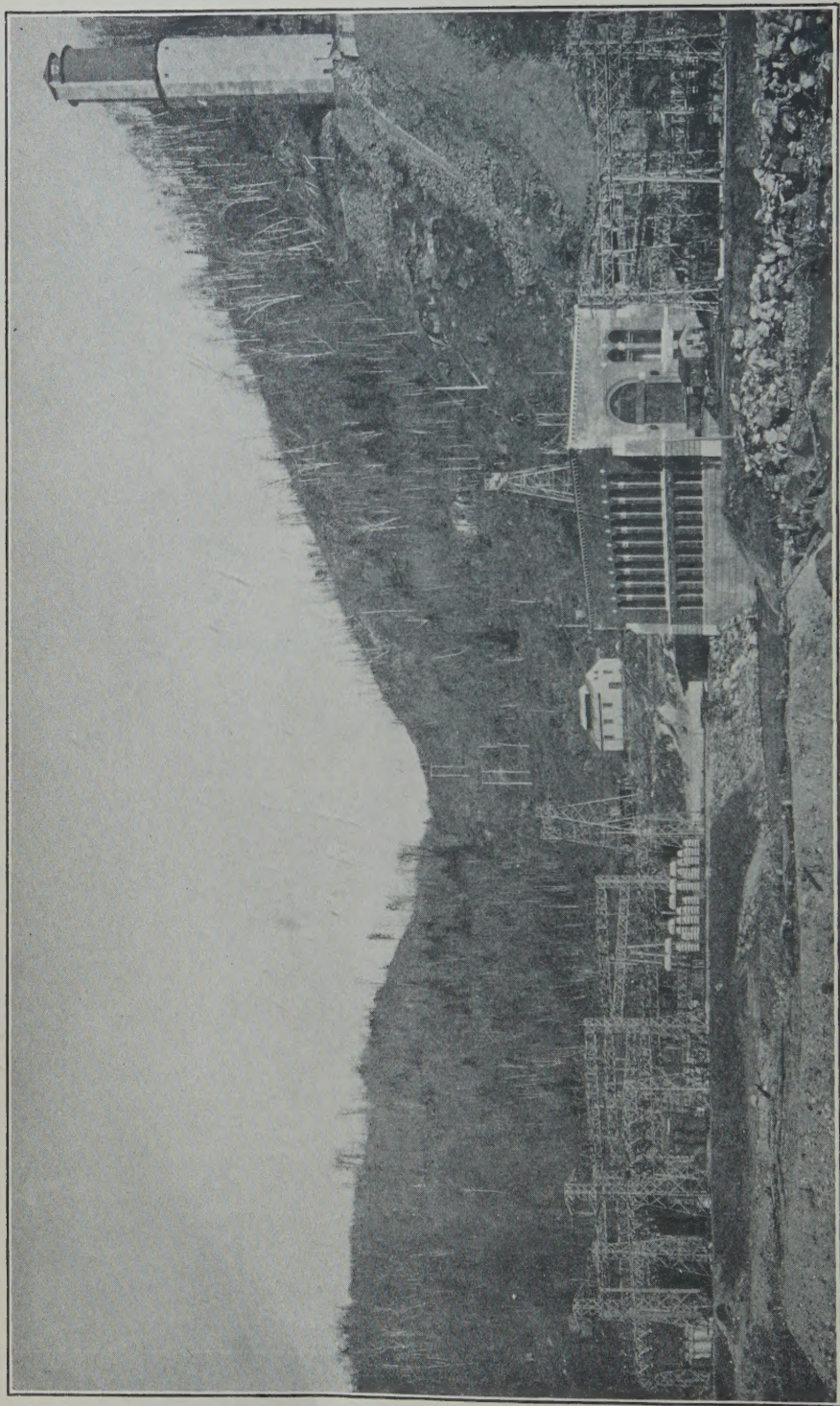




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WATER POWER ENGINEERING



Davis Bridge (Harriman) power and storage development of New England Power Company, power house and outdoor equipment. Three 20,000-hp. wheel units operating under a head of about 350 ft. at 360 r.p.m. (Courtesy of New England Power Company.)

(Frontispiece)

WATER POWER ENGINEERING

BY

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SECOND EDITION

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PREFACE TO THE SECOND EDITION

In the seven years that have elapsed since the preparation of the first edition important progress has been made in the field of water power engineering. The Kaplan adjustable blade propeller type turbine has come into wide use for lower heads. Transmission voltages have now reached 330 kv. and length of transmission 265 miles. The first pumped-storage peak load, water power plant in this country, at Rocky River, has been in use since 1928. Steam plants at public utility plants now average only 1.5 lb. of coal use per kilowatt-hour and large plants at tidewater can produce power at $\frac{3}{4}$ cent per kilowatt-hour and less.

Notwithstanding the lessened cost of steam power, there has been a rapid and wide development of water power as many advantages accrue from a substantial amount of water power in the larger load systems, and the low increment cost of water power makes it especially adapted for carrying peak load capacity. Large peak load water power developments have been built at Conowingo, Safe Harbor, and Fifteen Mile Falls. Water power and steam are now complementary in use rather than competitive.

The author has endeavored to bring all subject matter up to date and to include some important additional subjects. This has required the rewriting of much of Chap. XII to accord with progress in power economics, and a revision of the other chapters. The Group I problems at the end of the book are largely new and of greater scope. The Group II problems have not been changed as they are intended to illustrate types of problems which may be used for other rivers and localities, of more direct interest to students in different sections of the country.

Acknowledgments are made to the many who have made constructive suggestions and contributed information to the author for this revision.

H. K. BARROWS.

BOSTON, MASS.

July, 1934.

PREFACE TO THE FIRST EDITION

The increasing demand for power in all industries, its continually growing uses in the home, together with the high cost of coal and the serious interruptions in its supply due to recurring labor difficulties, have, in the past few years, focused public attention upon the need of comprehensive water power development. In response, many large water power projects have been constructed or are under way, particularly in the South and West.

Interconnection of power transmission systems is already well established and numerous large-sized economical base-load steam plants have been constructed near the great load centers.

It is so evident that all water power commercially available should be utilized that active development of such projects is likely to continue for some time.

The problems involved in water power design afford an admirable opportunity for study in applied hydrology, hydraulics, and mechanics, in recognition of which, as well as in response to the general interest in water power development, many of the technical schools now afford an opportunity for study in this field.

At the Massachusetts Institute of Technology a graduate course in Water Power Engineering has been given for several years, and since 1921 a hydroelectric option has been offered in Civil Engineering, including a course in Water Power Engineering extending throughout the fourth year. The need of an adequate textbook for these courses has led the author to prepare this volume.

The hearty and generous cooperation shown by engineers, operators, and manufacturers has been of the greatest assistance, and the author wishes here fully to acknowledge his indebtedness in this respect, as well as his deep appreciation of the many other courtesies rendered.

Special acknowledgment is due to Mr. William F. Uhl, Consulting Hydraulic Engineer, of Charles T. Main, Inc., Boston, who prepared Chap. X; to Mr. E. A. Graustein, Consulting Engineer of Boston, who prepared Chap. XI and rendered much valuable assistance upon other portions of the book; to Mr. R. A. Hopkins, Electrical Engineer of Stone & Webster, Inc., of Boston, who prepared Chap. VIII; to Prof. W. M. Fife of the Institute, who reviewed the greater portion of the manuscript; to the New England Power Company through Mr. Arthur C. Eaton, Hydraulic Engineer, New England Power Construction Company,

for plans and information; and to Mr. Robert E. Horton, Consulting Engineer of Albany, N. Y., for assistance.

While the author has aimed to produce a textbook in which the underlying principles are accentuated, still he hopes that he may have drawn from his practical experience in this field sufficiently to make the book of value to the practicing engineer.

H. K. B.

BOSTON, MASS.

December, 1926.

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WATER POWER ENGINEERING

CHAPTER I

WATER POWER DEVELOPMENT—ITS DISTRIBUTION AND USE HISTORICAL

The use of water power by crude devices dates back to ancient times. The primitive wheels, actuated by river current, were used for raising

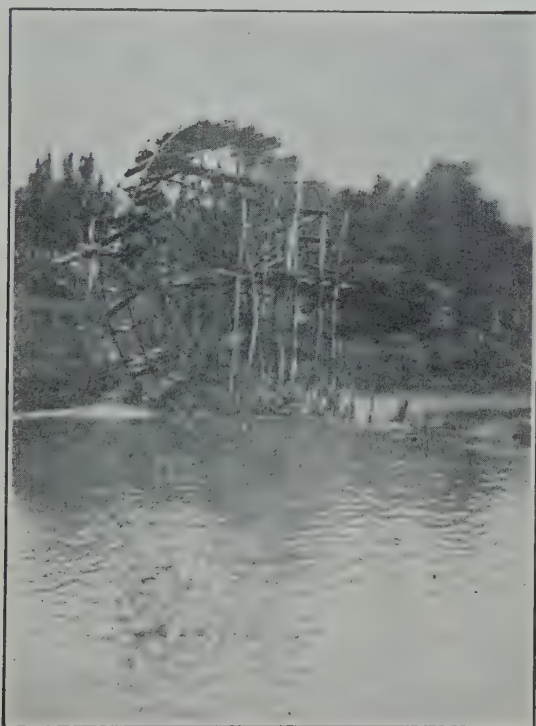


FIG. 1.—Chinese bamboo water wheel. (*Courtesy of Dr. C. K. Edmunds.*)

water for irrigation purposes, for mills in grinding corn, and in other simple applications. The Chinese nora or float wheel, built of bamboo, with woven paddles, is still in use, as well as other forms of current wheel elsewhere (see Figs. 1 and 2). Such devices have a very low efficiency and utilize but a small part of the power available in a stream.

The undershot, breast, and overshot wheels were great advances in that water was confined in a channel, brought to the wheel, and utilized under a head or fall. These were, therefore, gravity wheels and gave efficiencies from perhaps 30 per cent for the undershot wheel up to about 70 per cent for the breast, and 80 per cent or more for overshot wheels. The last were extensively used up to about 1850 and have been constructed and used to a limited extent since.



FIG. 2.—Water wheel in use near Murcia, Spain.

The development of the hydraulic turbine about the middle of the nineteenth century, however, resulted in its completely superseding the overshot wheel, the latter, while efficient, being of relatively large diameter and low speed and limited in head usually to from about 10 to 40 ft., this limit being imposed by the size of the wheels which could be constructed. The period between the middle and the end of the nineteenth century saw the development of the American, or mixed- or inward-flow turbine in this country, the evolution from the Francis wheel of 1849, to gain greater power and speed, and its gradual dominance over the outward-flow Boyden and Fourneyron wheels. The developments of these days were, however, limited to low heads, usually 15 to 20 ft., the use of a greater fall being made by canals in subdivisions, each a unit

of about 20 ft., as with the early developments of this time at Holyoke, Lawrence, and Lowell, Mass., Manchester, N. H., Cohoes, N. Y., etc.

About 1890 came electric transmission of power, greater stimulation of water power development, and its extension to single heads greater than 20 ft. At first this was met somewhat by the use of wheels designed in Europe but manufactured in America. Soon the development of the American type of wheel advanced to such an extent, however, that within a few years—by shortly after 1900—the range of developable head had increased to 100 ft. or more, and by 1910 American wheel manufacturers were building reaction wheels to use under heads of 500 ft. and upwards. The present maximum head for reaction wheels is about 1000 ft. in this country and 1180 ft. at the Zappello plant in Italy.

The impulse or Pelton wheel of the bucket and nozzle type was developed by 1890, especially for high-head use in the West, and in greatly improved form it still occupies this field, normally for heads above about 800 ft.

In both American and European practice, between 1900 and 1910, the desired power and speed were often obtained by the use of multiple-runner units, *viz.*, several wheels on one shaft. Since that time, however, the use of single-runner vertical wheel units has predominated, with better wheel design and the development of the vertical generator and suitable thrust bearings for large vertical wheel-generator units.

The war period of 1914–1918 and thereafter, with its great activities in manufacture, accompanied by great increase in cost of fuel and uncertainty of delivery on account of strikes and embargoes, had a marked effect on the desirability of water power development in this country, which still persists. Improvements in wheels and wheel settings, flumes, draft tubes, etc., have resulted in common working wheel efficiencies well over 90 per cent. Today American wheel manufacturers lead the world in all respects—size and capacity of wheel units, as well as efficiency—and the demand for hydroelectric power is increasing rapidly in all sections of the country where it is economically available.

The capacity and radius of practicable electrical transmission of power have also greatly increased, as shown by the following table showing transmission voltage at different times since 1900:

TRANSMISSION VOLTAGES, 1900–1933

Year	Voltage
1900.....	40,000
1910.....	110,000
1920.....	150,000
1924.....	220,000
1933.....	330,000

This increase in voltage has accompanied greater distance of transmission, so that in place of the few miles of the early days, lines 275 miles

long are now in use, and a length of 300 miles is entirely practicable at the present time. These long lines must be of large capacity, 50,000 to 100,000 kw. or more, however, to be economical for such distances.

Interconnection of power systems for better dependability, as well as higher load factors, has also proceeded, making it possible to tie in both steam and water power plants to their mutual advantage, and thereby decrease power cost.

The present activity and interest displayed in water power development are by no means confined to the United States. Canada is proceeding, even more rapidly, to develop its water power, both on account of the excellent opportunities for exploitation and her relatively greater lack of fuel for steam power.

Abroad rapid progress in water power development has occurred in the past decade—especially in Italy, France, and Japan.

For comparison with Figs. 1 and 2 a view of a large modern hydroelectric plant, with its outgoing high-tension transmission lines is shown in the frontispiece—the Davis Bridge (Harriman) plant of the New England Company, built in 1924–1925, one of the most important power and storage units in a large power system supplying southern New England.

AMOUNT OF WATER POWER

In studying the subject of water power, it will be of interest to consider first its world distribution, including both developed water power and that not yet utilized.

A comprehensive study of world water power was made in 1921 by the U. S. Geological Survey¹ giving much information. These estimates of power have been revised by the Survey in general to about 1930 and, for the United States and Canada, to Jan. 1, 1931.

Developed power is based upon installed wheel capacities and potential power upon flow at ordinary low water and a plant efficiency of 70 per cent. Installed wheel capacity often averages two or three times the potential low water power, so that in some countries the developed power equals or exceeds the potential.

For foreign countries all available published information was used. In Asia, Africa, and South and Central America, with much unexplored territory, the estimates had to be made as well as could be from studies of rainfall and topography, checking by such meager information as was available.

Even in the countries where the better information was available these estimates are rough, and this survey—the first of its kind for the world as a whole—gives of necessity only a very approximate idea of

¹ "Water Power of the World," Part II of "World Atlas of Commercial Geology."

water power resources, mostly of value in a relative way and for comparative purposes.

For the British Empire, "Hydroelectric Development in the British Empire," 1931, Macmillan & Co., Ltd., contains much information upon water power development.

General Distribution of Water Power.—In Table 1 is given the distribution of water power by continents:

TABLE 1.—WATER POWER BY CONTINENTS

Continent	Water power, million horse power		Per cent	
	Developed	Potential	Developed	Potential
North America.....	21.80	69	47.9	15
South America.....	0.90	44	2.0	10
Europe.....	18.40	56	40.4	13
Asia.....	4.00	71	8.8	16
Africa.....	0.03	190	0.1	42
Oceanica.....	0.37	17	0.8	4
Total.....	45.50	447	100.0	100

From the table it will be seen that for developed water power totaling about 45 mill. hp. nearly half is in North America and about 33 per cent is in the United States alone. Europe also has nearly 40 per cent of the developed water power of the world, so that the other continents all have relatively insignificant amounts.

For potential water power, including both developed and undeveloped, at minimum flow, the world total is estimated at about 447 mill. hp., with about the same amounts in each continent except Africa which has about three times the amount of any other continent, but practically no developed power. In fact, nearly half of the potential water power of the world lies in Africa and more than one-fourth in the Congo River basin alone.

Africa is essentially a great plateau on which the streams become large before they fall to the sea and fall rather abruptly. In northern and southern Africa rainfall is low and poorly distributed; hence, the available water power is small. But tropical Africa, particularly the Congo basin, has a heavy rainfall, which together with favorable topography produces the great potential water power noted above.

Asia, because of its vast area and high altitude of its central part, would have enormous water power except for deficient and irregular rainfall over most of its area. It accordingly ranks fifth of the continents.

According to Table 1 about 10 per cent of the world's potential water power is developed. Actually, based upon the ordinary range of wheel installations, which is far above the minimum flow used as a basis for the total potential power of 447 mill. hp., the present developed water power is only 4 or 5 per cent of the total potential power.

Compared to the total use of power in the world at present, the potential water power available is nearly four times as great in amount. It must be kept in mind, however, that a great part of this potential power is in wild or inaccessible regions, as illustrated especially by the vast African power resources and can perhaps never be utilized. On the other hand, the minimum power of streams can often be greatly increased by water storage, and by tying in with steam plants even the unregulated flow of a stream may often be profitably utilized for power for 6 to 9 months of the year. Hence, the lack of use of water power of the world on account of regional location is likely to be compensated largely by its greater use in countries well developed. The total of 447 mill. hp. of potential water power in the world is, therefore, far below the developable amount if it could all be used and probably not in excess of what may sometime be actually developed.

The relative distribution of water power by continents as regards area is shown in Table 2.

TABLE 2.—WATER POWER PER SQUARE MILE—BY CONTINENTS

Continent	Area		Water power, horse power per square mile	
	Thousands of square miles	Per cent	Developed	Potential (minimum)
North America.....	9,323	17.8	2.45	7.4
South America.....	6,889	13.1	0.13	6.4
Europe.....	3,879	7.4	4.75	14.4
Asia.....	17,057	32.6	0.23	4.2
Africa.....	11,515	22.0	0.00	16.5
Oceania.....	3,710	7.1	0.10	4.6
Total or average.....	52,373	100.0	0.87	8.6

For the United States alone (3,026,971 sq. miles), using the estimates of the U. S. Geological Survey for 1931 of about 15 mill. hp. developed and 38 mill. hp. of potential (minimum) water power (see page 30), the amounts per square mile would be:

	Horse power
Developed.....	4.95
Potential.....	12.5

For Canada (3,729,665 sq. miles) the corresponding figures would be:

	Horse power
Developed.....	1.64
Potential.....	4.9

From the foregoing it will be seen that while Europe as a continent leads, with about 4.75 developed hp. per square mile of area, the United States, with about 80 per cent of the area of Europe, materially exceeds the latter with about 4.95 hp. per square mile. Canada also is fairly well developed in water power.

For potential water power Africa, with 16.5 hp. per square mile, is in advance, with Europe second with 14.4 hp. per square mile. Here again the United States taken by itself ranks nearly with Europe, with 12.5 hp. per square mile.

Additional information upon water power resources is also available in the *Transactions* of the World Power Conference of 1924¹ and, for Canada, in the various reports of the Dominion Bureau of Statistics. The author is also indebted for information regarding European water power resources to A. J. R. Houston, gathered by him from various sources while studying abroad.

While the endeavor has been made to use the latest and best information available in discussing this subject, there are doubtless some discrepancies, due largely to a lack of standardization of data in the different countries. The American Engineering Standards Committee is, through a committee under the sponsorship of the U. S. Geological Survey, now preparing standard methods for the power rating of rivers, which should be helpful in future power estimates.

Water Power of the United States.—In 1924 estimates of water power by the U. S. Geological Survey were as follows:

United States	Total Water Power, 1924, Horse Power
Developed:	
Public utility and municipal.....	7,348,200
Manufacturing and miscellaneous.....	1,738,800
Total developed.....	9,087,000
Potential:	
Available 90 per cent of time.....	34,818,000
Available 50 per cent of time.....	55,030,000

Table 3, based upon estimates of the U. S. Geological Survey, shows the amount of developed water power by years subsequent to 1924 with the yearly increases.

¹ *Trans. of First World Power Conference*, vol. I, Percy Lund Humphries & Co., Ltd., London, 1924.

TABLE 3.—DEVELOPED WATER POWER IN UNITED STATES

Jan. 1 of year	Developed water power, horse power	Yearly increase	
		Horse power	Per cent
1926	11,176,596		
1927	11,720,983	544,387	4.9
1928	12,296,000	575,017	4.9
1929	13,571,530	1,275,530	10.4
1930	13,807,778	236,248	1.7
1931	14,884,667	1,076,889	7.2
1932	15,562,805	680,938	4.3
1933	15,817,941	255,136	1.6
Mean		663,450	5.0

The average increase in developed water power during the last 7 years (1926 to 1933 inclusive) has been about 663,000 hp. or about 5 per cent yearly. About 20 per cent of the nation's water power is in use at present, based upon a total developable amount of 80 mill. hp.

TABLE 4.—WATER POWER OF UNITED STATES BY DISTRICTS

District	Developed		Potential		
	Horse power	Per cent	Available 90 per cent of time, horse power	Available 50 per cent of time	
				Horse power	Per cent
New England.....	1,946,655	12.3	998,000	1,978,000	3.6
Middle Atlantic.....	2,395,766	15.2	4,317,000	5,688,000	10.4
East North Central....	1,139,209	7.2	737,000	1,391,000	2.5
South Atlantic.....	2,978,895	18.9	2,476,000	4,464,000	8.1
East South Central....	1,363,678	8.6	1,011,000	2,004,000	3.6
Total east of Mississippi River.....	9,824,203	62.2	9,539,000	15,525,000	28.2
West North Central....	767,368	4.9	871,000	1,844,000	3.4
West South Central....	149,551	0.9	434,000	888,000	1.6
Mountain.....	1,211,945	7.6	10,736,000	15,513,000	28.2
Pacific.....	3,864,774	24.4	13,238,000	21,260,000	38.6
Total west of Mississippi River.....	5,993,638	37.8	25,279,000	39,505,000	71.8
Grand total.....	15,817,941	100.0	34,818,000	55,030,000	100.0

Distribution of Water Power by Districts.—In Table 4 is given water power for the main divisions of the country as determined by the U. S. Geological Survey, for potential power in 1924 and for developed power as of Jan. 1, 1933.¹

As will be seen from the foregoing table, roundly speaking about 60 per cent of the developed water power of the United States is east of the Mississippi River (mostly relatively near to the coast) and 70 per cent of the potential water power lies west of the Mississippi and nearly all of this in and west of the Rocky Mountains. Nearly 40 per cent of the potential power is in the Pacific states alone—California, Oregon, and Washington.

Furthermore, as a district the Pacific states now also lead in the amount of developed power with 24.4 per cent of the total.

The average size of development is much greater in the newer developments of the South and West than in New England and the North, as shown by the following:

TABLE 5.—AVERAGE SIZE OF WATER POWER DEVELOPMENTS

District	Total capacity, horse power	Number of plants	Average size of plant, horse power
New England.....	1,946,655	1,198	1,630
Middle Atlantic.....	2,395,766	579	4,140
South Atlantic.....	2,928,995	326	9,150
Pacific.....	3,864,774	307	12,500

The estimates of potential power in Table 4 are based upon 70 per cent overall efficiency at both developed and undeveloped sites. On the Colorado River it is assumed that stored water will be used to equalize

TABLE 6.—LEADING STATES IN WATER POWER

Rank	Developed		Potential (available 50 per cent of time)	
	State	Horse power	State	Horse power
1	California.....	2,447,736	Washington.....	7,871,000
2	New York.....	1,900,060	Oregon.....	6,715,000
3	Washington.....	1,053,206	California.....	6,674,000
4	North Carolina.....	970,860	New York.....	4,960,000
5	Alabama.....	931,450	Idaho.....	4,032,000

¹ "Power Capacity and Production in the United States," U. S. Geological Survey, *Water Supply Paper* 579.

flow below the mouth of Green River. One-half the potential power of the Niagara River and the international section of the St. Lawrence River are also included.

Water Power by States.—Considering individual states, those leading in water power are shown in Table 6.

In general, the minimum or 90 per cent flow power is about two-thirds of that available 50 per cent of the time. Notable exceptions to this occur in New York, where this relation is about 80 per cent, and Arizona with 95 per cent, due to the equalized flow of the Niagara and St. Lawrence rivers in New York and of the Colorado River in Arizona.

Relation of Present and Future Development.—The total potential power of about 55 mill. hp. available 50 per cent of the time does not represent the ultimate installed capacity of wheels. Based upon the relation of these two quantities, using data for New England (omitting Rhode Island and Connecticut), where capacity of wheels at developed sites is about 131 per cent of the power available 50 per cent of the time, the total installed capacity of wheels may ultimately reach over 72 mill. hp. The present installed capacity is, therefore, about one-fifth of the potential water power of the country, as a rough estimate.

Relation of Water Power to Total Power Use.—The estimated capacity of stationary prime movers in the United States, based upon reports of the U. S. Geological Survey and the U. S. Census of Manufactures, has been as follows, as compared with that obtained from water power:

TABLE 7.—CAPACITY OF STATIONARY PRIME MOVERS IN THE UNITED STATES

Year	Million horse power		Water power as per cent of total
	Total	Water power	
1900	17.5	3.3	18.8
1905	24.0	4.4	18.3
1910	32.5	6.0	18.4
1915	40.2	7.8	19.3
1920	49.0	9.5	19.4
1925	60.0	10.5	17.5
1930	69.0	13.8	20.0

About one-fifth the total power used is therefore water power, based on installed capacity, and this ratio has changed but little in the past 30 years.

Annual Power Output.—Since 1919 the U. S. Geological Survey has been receiving and compiling monthly reports of power production from public utility plants, including 95 per cent of the plants and estimates for the remainder, which include public power and lighting and

railway use, but not power generated at manufacturing plants. A summary of these results is as follows:

TABLE 8.—ANNUAL PRODUCTION OF POWER BY PUBLIC UTILITY PLANTS IN THE UNITED STATES

Year	Annual production, billion kilowatt-hours			Average coal consumption per kilowatt-hour, pounds
	From water power	From fuel power	Total	
1919	14.6	24.3	38.9	3.2
1920	16.2	27.4	43.6	3.0
1921	15.0	26.0	41.0	2.7
1922	17.2	30.4	47.6	2.5
1923	19.3	36.3	55.6	2.4
1924	20.0	39.0	59.0	2.2
1925	22.3	43.5	65.8	2.1
1926	26.2	47.6	73.8	1.95
1927	29.9	50.3	80.2	1.84
1928	34.7	53.1	87.8	1.76
1929	34.6	62.7	97.3	1.69
1930	33.0	62.9	95.9	1.62
1931	30.5	61.2	91.7	1.55
1932	34.0	49.1	83.1	1.50
1933	35.5	50.0	85.5	1.47

The increase in power production by periods was as follows:

Period	Item	Yearly increase, billion kilowatt-hours	Yearly increase, per cent
1919-1929	Water power	2.1	9.4
	Total power	5.8	9.8
1929-1933	Water power	0.8	2.8
	Total power	-3.5	-3.6
1919-1933	Water power	1.7	7.9
	Total power	3.7	6.7

The yearly production of total power fell off after 1929 to a minimum in 1932 (which was about 85 per cent of the 1929 production) and until May, 1933, since which time it has again increased, the year 1933 showing about 89 per cent of 1929 production and about 2.5 per cent more than in 1932.

The yearly production of water power fell off in 1930-1931 due to both the business depression and general low water conditions but increased markedly in 1932 and in 1933 was about 4 per cent greater than in 1929.

The total output of electrical power is now regarded as a useful index of business conditions and an index based upon this is reported each week by the Edison Electric Institute of New York.

The total generator capacity in 1929 was about 29,600,000 kw. (a ratio of output to capacity for that year of about 0.37) corresponding to an engine or wheel capacity of about 41 mill. hp. or a little over half the total capacity of stationary prime movers as given in Table 7.

In the last column of Table 8 is given the average coal consumption per kilowatt-hour of output for the public utility plants. The use of coal in a period of approximately 10 years has been cut in half, indicating the great improvement effected in equipment and operation. The "average" plant is now using about 1.5 lb. of coal per kilowatt-hour, which means that many plants are below this figure.

The leading states in power production for the year 1929 were as follows:

TABLE 9.—ELECTRIC POWER PRODUCTION IN THE UNITED STATES, 1929—LEADING STATES

Rank	Water power			Fuel power			Total power		
	State	Total billion kilowatt-hours	Per cent of total in the United States	State	Total billion kilowatt-hours	Per cent of total in the United States	State	Total billion kilowatt-hours	Per cent of total in the United States
1	California.....	6.366	18.4	New York....	8.847	14.1	New York....	14.465	14.9
2	New York....	5.617	16.3	Pennsylvania..	7.323	11.7	Pennsylvania..	8.154	8.4
3	North Carolina	2.373	6.9	Illinois.....	7.113	11.4	California....	8.849	9.1
4	Washington...	2.182	6.3	Ohio.....	6.372	10.2	Illinois.....	7.369	7.6
5	South Carolina	1.316	3.8	Michigan....	3.595	5.7	Ohio.....	6.417	6.6
Totals.....		17.854	51.7		33.250	53.1		45.254	46.6

As will be seen from the above table, about half the production of electric power in the United States is in the leading five states. New York is well ahead in total power and also first in fuel-power use (having about the same output as Pennsylvania in this respect) and is second only to California in respect to water power. The importance of the states in manufacturing activity is shown best perhaps by their use of fuel power, although it must be kept in mind that power used directly in manufacturing is not included in these production figures.

Relative Economy of Water Power in the United States. Variation Due to Natural Conditions.—The sections of this country, relatively speaking, which are particularly endowed with water power facilities are in general the coastal mountain regions (including also the area

tributary to the St. Lawrence River), where sufficient fall is found in the rivers to make them feasible for power development. The other natural conditions, geologic and topographic, are also most favorable in these sections, particularly in the North and East, where the deposits of the glacial period have had a marked effect in changing the courses of rivers and their tributaries, so that in many places in excavating the new courses ledges and ridges of rock were encountered, thus developing falls and rapids which now afford great water powers. The deposits laid down in glacial times also resulted in numerous lakes and swamps which now afford excellent opportunities for storage developments.

The predominance of metamorphic rocks in New England gives rise to a greater number of concentrated falls than anywhere else on the Atlantic slope. Abrupt falls are less frequent in the southern states, the falls generally occurring as long shoals or rapids, with river bed of gravel or boulders. The southern streams, while free from ice freshets, are more irregular in flow than those of the Northeast, due both to more general lack of storage facilities and more irregular occurrence of rainfall.

The rivers tributary to the St. Lawrence, especially in New York, compare favorably in respect to conditions favoring power development with those of the Atlantic slope, although in general in the former district the rocks are softer and foundation conditions less advantageous.

In the central states, in general, river slopes are moderate, concentrated falls lacking, and conditions do not favor water power development. Curiously enough, however, as an exception to the general rule, it is in this region that one of the largest single power developments is found in the United States, *viz.*, at Keokuk—a plant of 170,000 hp. so laid out that eventually its capacity can be doubled.

In the prairie states, while there are some large streams, conditions of river slope and bed do not favor power developments and flow is very irregular and subject to a wide variation in its extremes.

The area including the Pacific coast and slopes is the great potential water power district of the country, particularly in California, Oregon, and Washington, and here again conditions of fall and river bed generally favor power development. In many cases, too, regularity of flow is aided by snow and glacial storage in the elevated regions tributary to this district.

Another factor of importance in providing a well-sustained seasonal stream flow in certain western locations is the ground storage available in porous volcanic deposits.

Variation Due to Regional Conditions.—As has been stated elsewhere (page 9), approximately 60 per cent of the developed water power of the country lies east of the Mississippi River (mostly near the coast), while 70 per cent of the potential power lies west of this river and is mostly in the Pacific coast states.

The early development of water power in New England and New York was due partly to favorable natural conditions for development and partly to the demand for power in manufacturing. The more desirable power sites in these districts are therefore largely utilized, although in the northerly and more distant portions there still remains considerable undeveloped water power, which will doubtless be utilized as the present network of transmission lines becomes more extended. The various investigations for the Superpower District (see page 34) indicated the desirability of utilizing all water power in the Northeast as soon as practicable.

In the South there is also a good market for power, especially in cotton manufacture, and the water powers of the South Atlantic slope are likely to be quite fully utilized.

While great strides have been made in water power development in the Pacific coast states in the last decade, there is still a great surplus of undeveloped water power. Fowler¹ estimated the present and future power needs of this district as follows:

TABLE 10.—POWER REQUIRED IN PACIFIC COAST STATES

Area or market	Mean yearly horse power required		
	1921	1930	1940
Puget Sound.....	109,600	253,400	572,900
Eastern Washington.....	65,800	129,200	242,500
Idaho—Utah.....	102,500	230,900	476,100
Portland.....	62,900	141,800	292,200
North California.....	303,900	533,200	868,500
South California.....	306,300	880,200	2,083,000
Total.....	951,000	2,168,700	4,535,200

As the minimum potential water power of this district is over 13 mill. hp. (see page 8), there is evidently ample potential water power in this district to suffice for many years. Owing to the comparative scarcity of good steam coal in these coastal states (Washington being the only one of these seven states where it has been used to any great extent in power generation) and high cost of other fuels, water power is bound markedly to predominate in the future power use in this part of the country.

Notable Water Power Plants and Sites in the United States.—New England has some of the oldest water power developments in the country constructed for manufacturing purposes, including:

¹ FOWLER, F. H., "Water Power Potentialities of the Pacific Coast," *Trans. A.S.C.E.*, p. 836, 1923.

	Horse Power
Holyoke.....	50,000
Lowell and Lawrence.....	25,000 each
Manchester.....	30,000
Lewiston.....	25,000

Of these Holyoke especially produces paper, while the other cities have large cotton and woolen mills. These developments are nearly all of the old type with two or more canal levels, except that at Manchester, a development of 15,000 hp. (built 1921–1922) with space for another unit of 7500 hp., utilizing the full head at the privilege, has partially superseded the older development.

The largest single development in New England is at Fifteen Mile Falls on the upper Connecticut River, constructed during 1929–1930, with 215,000 hp. of wheels in four units under a head of about 170 ft.

In Vermont the Silver Lake development of the Horton Power Company utilizes a head of about 576 ft. with 4000 hp. at turbine shaft—the highest head utilized in the Northeast.

Storage development has proceeded extensively in New England and many streams have well-sustained flow, notably the Penobscot, Kennebec, Androscoggin, and Presumpscot in Maine, the Merrimack, and the Deerfield.

The last river has been very extensively developed for power and storage by the New England Power Company. Storage reservoirs totaling about 8 bill. cu. ft. capacity have been constructed, a total head of about 1100 ft. has been developed in seven plants aggregating about 115,000 hp. of capacity, and there remains some 600 ft. of undeveloped fall in five or six privileges. This company is now supplying power to many localities in southern New England.

New York is another locality where some of the earlier water power developments were built. At Cohoes on the Mohawk River, a cotton-mill city, was an early development of the old type which has within the last few years been redeveloped in one fall; with a plant of 30,000-hp. capacity. About 200,000 hp. of water power is used in New York State in the manufacture of paper.

Excellent opportunities for storage are found in the New York streams, and under a comparatively recent law “storage-regulating districts” may be formed under state supervision to develop and assess the construction and operating costs of storage projects. Thus far such districts have been formed on the Black River and the Hudson River; in the latter case a large storage project on Sacandaga River, the principal headwater tributary of the Hudson, began operation in 1931.

Niagara Falls is perhaps the most noted power site in the world, with a total fall to below the whirlpool of about 310 ft. and some 6 mill. hp.

for the full river flow (averaging about 212,000 sec.-ft.), but only about 25 per cent of this, or 1.5 mill. hp., is authorized by treaty to be developed.

The present largest development in America, the Chippewa development of the Hydroelectric Commission of Ontario, with a total capacity of 525,000 hp. in 10 units, is at Niagara.

On the St. Lawrence River below Lake Ontario the fall is as follows:

	Feet
In 113 miles adjacent to New York State.....	91
In 70 miles between New York State and Montreal.....	130
Total.....	221

The mean river flow is about 221,000 sec.-ft., and the potential power of this stretch of river as estimated by the Dominion Water Power Branch is about 3 mill. hp., of which about 1 mill. hp. is on international waters. In this latter stretch about 75,000 hp. is developed at Massena, N. Y., for the production of aluminum. In the portion of the river wholly in Canada, about 200,000 hp. is developed at Cedar Rapids, etc., mostly used in Montreal. The use of this St. Lawrence power in New England and New York is now under serious consideration as a combined navigation and power project, as it is all within transmission distance of these districts. A further description of the power possibilities of the St. Lawrence and of Quebec is given on page 21.

In the other Middle Atlantic states much water power is developed, although conditions do not favor water storage as well as in New York and New England. The Conowingo development on the Susquehanna River in northern Maryland, completed in 1928, is the largest plant in this district, with seven units totaling about 378,000 hp. under a head of about 90 ft. (with an ultimate capacity of about 600,000 hp.). The Safe Harbor, Pa., plant of the Pennsylvania Water & Power Company completed during 1931, located upon the Susquehanna River just above the Holtwood plant, has six Smith-Kaplan units totaling 255,000 hp. under a head of 53 ft. The ultimate capacity of this plant will be double its initial capacity. A large undeveloped power site is at the Great Falls of the Potomac near Washington, a stream, however, of much fluctuation in flow.

The South is an important water power district fast developing. The seven large power companies in the five states, the Carolinas, Georgia, Alabama, and Tennessee, in this district have an approximate present capacity of water and steam plants of about 3 mill. hp. as follows:¹

¹ SAVILLE, THORNDIKE, "The Power Situation in the Southern Appalachian States," *Mfrs. Rec.*, Apr. 21, 28, 1927.

TABLE 11.—POWER DEVELOPMENTS IN THE SOUTHEAST

Company	Capacity, horse power		
	Hydroelectric plants	Steam plants	Total
Southern Power Company.....	830,000	250,000	1,080,000
Alabama Power Company.....	610,000 ^(a)	210,000	820,000
Tennessee Electric Power Company.....	242,000	120,000	362,000
Georgia Power Company.....	330,000	30,000	360,000
Carolina Power and Light Company.....	70,000	50,000	120,000
Central Georgia Power Company.....	24,000	60,000	84,000
Columbus Electric and Power Company.....	72,000	12,000	84,000
Total.....	2,178,000	732,000	2,910,000

^a Including Wilson Dam, 260,000 hp.

The transmission lines of these companies form an interconnected network covering much of the area of these five states, and in 1925 over 4 bill. kw.-hr. of power was generated by them. The power load is mainly industrial, largely cotton mills, with some mines, iron and steel mills, etc.

One of the largest plants is that of the Georgia Railroad and Power Company at Tallulah Falls, built in 1912-1913, now of 108,000-hp. capacity in six units operating under a net head of about 590 ft. The Tugalo plant of this company completed in 1924 has 88,000 hp. of wheels in four units under a 149-ft. head.

The Southern Power Company in 1923 completed the Mountain Island plant of about 91,000 hp. in four units under an 80-ft. head and since then the Badin plant of 121,000-hp. capacity.

The Alabama Power Company also has some large developments, notably the Lock 12 development on the Cross River, of 109,000 hp. in six units under a 70-ft. head, at the Mitchell dam of 72,000 hp. in three units under a 70-ft. head, and at the Martin dam of 135,000 hp. under 145-ft. head.

The Muscle Shoals development at the Wilson dam on the Tennessee River, with eight units under a 92-ft. head, totaling 260,000 hp., is planned with an ultimate installation of 612,000 hp.

The central and prairie states are relatively unimportant as respects water power, as the rivers in general are of slight fall and not well adapted for development. Some fair-sized plants are found in Wisconsin and Minnesota, and the Keokuk plant on the Mississippi, as already noted, is of exceptionally large capacity. The diversion of water (some 10,000 sec.-ft. or more) from Lake Michigan through the Chicago Drainage Canal to the Illinois River has also created a number of good power sites at Lockport, Marseilles, etc., in Illinois.

In the mountain states considerable water power is available in the headwater streams, notably the upper Missouri River, in the general vicinity of Great Falls, Mont., where six plants aggregating 220,000 hp. have been developed, including power used for traction by the Chicago, Milwaukee, St. Paul and Pacific Railroad in this district. In the Great Basin between the Rocky Mountains and the Sierra Nevada range are a number of developments and opportunities for water power, mainly in the Wasatch range in Utah and Idaho. This is, however, a region utilizing but relatively little power.

The Colorado River basin has received much recent publicity and is of great importance for both irrigation and power. The most important potential powers are between the mouth of the San Juan and the mouth of Black Canyon, the total fall between the two points being over 2500 ft. There are, however, some large water powers in the upper basin on the two branches, particularly on Green River. Estimates of capacity of the various power projects on the Colorado River vary greatly depending upon the assumed manner of development and extent to which the river flow is to be regulated. According to the 1921 *Report* of the U. S. Geological Survey, the potential power at 18 of the largest sites has been estimated at anywhere from 2 to 6 mill. hp. depending on the method of development.

The eventual chief market for the power of the lower Colorado River is southern California. Kelly¹ estimated power available below Diamond Creek and the Grand Canyon at from about 600,000 to 1,000,000 hp. (with upriver storage and allowance for irrigation use) depending upon the manner of development. He further suggested an initial plant capacity of 300,000 kw., which could be ready for use by 1930-1932, at which time it could be used in the Arizona and California market.

The development of the Boulder Dam project upon the Colorado River at the Black Canyon site between Nevada and Arizona, near Las Vegas, Nev., for power, irrigation, and flood control was authorized in 1930 and is now rapidly proceeding under the U. S. Bureau of Reclamation. This includes a curved gravity section concrete dam about 730 ft. high and provision for fifteen 50-cycle main power units, each of 115,000 hp. under about 500 ft. head at 150 r.p.m., and two 40-cycle units of 55,000 hp. each. The initial installation now proceeding includes four of the large units and one of the smaller units.

In the state of California many large water power developments have been recently constructed, including those of the Southern California Edison Company and subsidiary companies in central and southern California and of the Pacific Gas and Electric Company and the Great Western Power Company farther north.

¹ KELLY, WILLIAM, "The Colorado River Problem," *Trans. A.S.C.E.*, 1925 p. 306.

Among the interesting power developments in California is the Caribou development on the North Fork of Feather River, constructed 1920-1921 (see page 204), with 90,000 hp. in three units of impulse wheels, each of 30,000 hp. operating under a head of 1008 ft.—until recently the largest powered units for wheels of this type in the country but now exceeded by the 56,000-hp. unit at the Big Creek plant 2A of the Southern California Edison Company, under a head of 2200 ft.—the highest developed in this country.

In Oregon and Washington, there are many excellent sites for large developments. The Oak Grove development, near Portland, is noteworthy for the use of reaction wheels under a head of about 950 ft.

The Columbia River is one of the outstanding streams in the country in its possibilities for power, navigation, and irrigation development. It is 1250 miles long, with 750 miles in the United States, and between Snake River and Warrendale (about 30 miles above Portland) there is a fall of about 309 ft. in 184 miles. Studies of power development of this section of river by the U. S. Engineer Corps¹ indicate that from 1 mill. to about 2.5 mill. kw. of firm power can be developed, depending upon the manner of development—the largest suggested project, at “The Dalles,” requiring 38 units of 100,000 kw. each under a head of about 266 ft. On the Deschutes River, one of the tributaries of the Columbia, in central Oregon there are many excellent power sites capable of development at relatively low cost.

Recent large developments in Washington have included the Diabolo project for Seattle (to be completed in 1934) of 166,000-hp. capacity (ultimate 320,000 hp.) under a head of 310 ft. (see page 205), and the Rock Island project of the Puget Sound Light and Power Company on Columbia River (1931) of 84,000-hp. capacity (ultimate 210,000 hp.) under a head of 32 ft.

Water Power of Canada.—The comprehensive development of water power in Canada is in general on a more definite basis than in the United States, as so much of the water power is controlled by the government. This control is direct in the case of Manitoba and the provinces westerly thereof but is vested with the provincial authorities (who cooperate with the central government in work of investigation) in the rest of the Dominion. Studies in detail have been made of most of the important water powers in Canada by the Dominion Water Power Branch, and all new developments must be consistent with their plans, thus assuring a logical and eventually a complete development, as far as practicable, of all power streams. Incidentally, a hydrometric survey staff is employed in systematic and continuous stream measurements and the compilation and publication of stream-flow data.

¹ “The Lower Columbia River Project,” *Military Eng.*, January-February, 1933, pp. 38-44.

The water power of Canada is one of its basic and most valuable natural resources, and the capital invested in its development, of over a half billion dollars, makes it also one of the greatest single industries of the Dominion.

In Table 12 is the available and developed water power by provinces, as given in the *Report* of the Dominion Bureau of Statistics, Jan. 1, 1933.

TABLE 12.—AVAILABLE AND DEVELOPED WATER POWER IN CANADA
Jan. 1, 1933

Province	Available 24-hr. power at 80 per cent efficiency		Turbine installation, horse power
	At ordinary minimum flow, horse power	At estimated flow for maximum development, dependable for 6 months, horse power	
British Columbia.....	1,931,000	5,103,500	713,792
Alberta.....	390,000	1,049,500	71,597
Saskatchewan.....	542,000	1,082,000	42,035
Manitoba.....	3,309,000	5,344,500	390,925
Ontario.....	5,330,000	6,940,000	2,208,105
Quebec.....	8,459,000	13,064,000	3,357,320
New Brunswick.....	68,600	169,100	133,681
Nova Scotia.....	20,800	128,300	112,167
Prince Edward Island.....	3,000	5,300	2,439
Yukon and Northwest Territories...	294,000	731,000	13,199
Total.....	20,347,400	33,617,200	7,045,260

The basis for power estimates in the above table is similar to that used by the U. S. Geological Survey (see page 4), except that a wheel efficiency of 80 per cent is used in the Canadian estimates instead of 75 per cent as in the United States. An analysis of existing plant installation in Canada indicates an average capacity about 30 per cent

TABLE 13.—USE OF DEVELOPED WATER POWER IN CANADA, 1930

Use	Horse power	Per cent
Central stations.....	5,150,000	84
Pulp and paper mills.....	580,000	9.5
Other industries.....	395,000	6.5
Total.....	6,125,000	100

greater than the 6-months-flow horse power, and, if this relation continues, it would mean that the total ultimate turbine installation would be nearly 44 mill. hp. The present installation of turbines, of about 6.7 mill. hp., represents, therefore, only about 15 per cent of the total water power resources.

The use of developed water power in Canada is shown in Table 13.

As will be noted from the above, 84 per cent of the water power is used in central stations which develop power for sale, and 10 per cent in the pulp and paper-mill industry—the most important industrial use. Over a third of the motors driven by the central stations are those in pulp and paper mills.

The development of water power in Canada is proceeding rapidly and during the 5 years, 1926–1931, new plant installations totaled about 2.3 mill. hp.—an increase of over 50 per cent of the capacity in 1926. Some of the larger recent installations are listed in Table 14.

The great development at Niagara—the Queenstown-Chippewa project of the Hydroelectric Power Commission of Ontario already referred to on page 16 is notable not only for size of units but also as the largest capacity single plant in America—10 units totaling 550,000 hp. La Loutre reservoir on the St. Maurice River in Quebec, with a capacity of 160 bill. cu. ft., is one of the largest reservoirs for power use yet constructed.

The Saguenay River in the province of Quebec below Lake St. John has a fall of over 300 ft. to tidewater at Chicotimi. Lake St. John with a water area of 350 sq. miles and a useful storage capacity of 220 bill. cu. ft. makes possible a regulated flow of about 35,000 sec.-ft. in Saguenay River. In 1925 the Duke-Price interests developed the Isle Maligne project about 7 miles below Lake St. John, with a head of about 100 ft. and capacity of 540,000 hp. The remaining fall, which will give about 1 mill. hp. more power, has an initial temporary development by the Alcoa Power Company at Chute à Caron of 260,000 hp. under a head of 150 ft. constructed in 1927–1931, which will be ultimately replaced by the Shipshaw development, which will probably have the power house built in several stages of 200,000 to 300,000 hp. each and when completed will be one of the largest of its kind.¹

Water power development in the vicinity of Winnipeg on the Winnipeg River is also proceeding rapidly, this being one of the important power streams of the continent due to the magnificent storage afforded by the Lake of the Woods. The ultimate development in this vicinity will be over 400,000 hp., all within easy transmission distance of Winnipeg and mostly at very low cost.

¹ "The Chute à Caron Hydroelectric Development," *Proc. Eng. Soc. W. Pa.*, January, 1931.

TABLE 14.—RECENT LARGE HYDROELECTRIC DEVELOPMENTS IN CANADA¹

Plant	Company	River	Province	Date of construction	Capacity, horse power	Number of units	Head, feet	R.p.m.
Rapide Blanc.....	Shawinigan Water and Power Company	St. Maurice	Quebec	1930-1932	160,000	4	108	109.1
Beauharnois.....	Beauharnois Light, Heat and Power Company	St. Lawrence	Quebec	1929-1932	212,000	4	80	75
Chute-à-Caron.....	Alcoa Power Company, Ltd.	Saguenay	Quebec	1927-1931	260,000	4	150	120
Chats Falls.....	Hydroelectric commission, Ontario	Ottawa	Quebec and Ontario	1929-1931	224,000	8	53	125
Abitibi.....	Ontario Power Service Corporation	Abitibi	Ontario	1930-1933	330,000	5	237	150
Seven Sisters.....	North Western Power Company, Ltd.	Winnipeg	Manitoba	1929-1931	112,500	3	66	138.1
Masson.....	McLaren Quebec Power Company	Lievre	Quebec	1931-1933	136,000	4	185	166.7

¹ *Power*,—Jan. 5, 1932, p. 24; January, 1933, p. 22.

Canada is well in advance in water power development as regards population, there being about 708 hp. per thousand of population in 1932. As further noted, this figure is surpassed only by that of Norway, with about 715 hp. per thousand, while the United States has about 129 hp. per thousand.

Water Power of Central and South America.—In Central America, while there is a considerable amount of potential water power due to plentiful rainfall and numerous power sites, there is as yet relatively little power development, owing to lack of manufacturing and no demand for power except for municipal use in the larger cities.

In South America, the great mountain system of the Andes so divides the drainage of the continent that but a relatively small portion is tributary to the Pacific. The rivers in this district, while short, have a great fall and in many cases extend to the heights of perpetual snow. Little water power development has been made as yet except for municipal lighting and power.

Some of the great rivers of the western world lie to the east of the Andes and afford great potential water power, particularly the Parana, which with the Uruguay River forms the La Plata. The Amazon, owing to its vast size, also has great potential power.

The Parana, with its tributaries, is the great power stream of South America. At Maribondo on the Rio Grande (one of the principal tributaries of the Parana) near Sao Paulo, Brazil, is a site with power estimated at 600,000 hp. The Parana where it forms the boundary between Brazil and Paraguay has at Sete Quedus (seven falls) a fall of some 375 ft. and a potential power estimated at 20 mill. hp. or more. On the Iguassú, another tributary of the Parana, near its entrance to the latter, are the great Iguassú Falls, 230 ft. high, and the famous Kaieteur Falls on the Potaro River in British Guiana have a total drop of 820 ft.

Of the 44 mill. potential horse power available in South America, however, of which about one-half is in Brazil, only about a million has been developed as yet.

Water Power of Europe.—As previously noted, Europe has about one-third of the total developed water power in the world, ranking next to North America, which has about one-half. In potential water power per square mile Europe also ranks high, being second only to Africa in this respect. Rainfall in general is plentiful, but much of the continent is less than 1000 ft. above sea level. The mountainous areas are in the Scandinavian peninsula and along the eastern edge of European Russia, and in the south, the Caucasus Mountains, the Alps, and the Pyrenees.

Norway leads the various countries of the world in respect to both developed and potential water power per thousand of population and ranks third in developed horsepower per square mile of area. The utilization of electricity in Norway is most highly developed, some 75 per

cent of the population being provided with electricity and 10 per cent of the natural wealth represented by hydroelectric installations. Coal is almost wholly lacking.

Recent government estimates show about 12 mill. hp. of potential power and about 2.3 mill. hp. developed. On the lower Glommen River, in Ostfold, there are four sites having a total capacity of 300,000 hp., of which 188,000 hp. is developed. At Rjukan on Maane River are two developed sites totaling about 239,000 hp. Utilization of water power for chemical industries in Norway is particularly noteworthy, and plants from 100,000 to 200,000 hp. have been installed for producing nitric acid from atmospheric nitrogen, as the chief water powers of the Norwegian sites (which have concentrated fall and large volumes of water) are well adapted for this industry. These powers are also well located near the head of tidewater, convenient for ocean freightage. Water power has been developed in both Norway and Sweden to some extent by the state.

Sweden.—The rivers of Sweden are not large, but are naturally favorable for storage. Winter temperatures are sufficiently low to require precautions against anchor and frazil ice but not low enough to prevent the utilization of water power even in the extreme north. Since 1906 the state has taken an active part in water power development, and, under the administration of the Royal Water Fall Board, plants have been erected at Trollhattan on the Lota a short distance below Lake Venern (155,000 hp.), at Elfkäleo (75,000 hp.), and other locations. The government is also electrifying the railroads. The potential power of Sweden is estimated at about 5 mill. hp., with about 1,700,000 hp. developed.

Russia.—At the outbreak of the World War, Russia in both Europe and Asia comprised over 8 mill. sq. miles, or about one-seventh of the land area of the globe, of which approximately 2 mill. sq. miles was in Europe. The new states formed since the war—Finland, Poland, etc., in the west, and the Ukraine, Georgia, etc., in the south—contain most of the water power resources of former Russia. In European Russia in general there is comparatively little rainfall and it is poorly distributed for water power use. It is only the great size of the drainage basins due to the even relief that makes the rivers of Russia the largest in Europe. The total potential water power in present Soviet Russia is estimated at 8 mill. hp. in European Russia and 8 mill. hp. in Siberia, with but little developed power. The Dnieprostroy hydroelectric development, completed in 1932 in the Ukraine, upon the Dnieper River, has a total capacity of 756,000 hp. in nine units, under a head of 116 ft. and is the largest powered single plant as yet developed.

Finland has about 1.8 mill. hp. of potential water power and about 250,000 hp. in use. The use of its water power is restricted in many cases by low temperatures prevailing for a considerable part of the year.

Poland has about 1.4 mill. hp. of potential water power, largely on the Vistula River but only about 90,000 hp. developed.

Germany has about 2 mill. h p. of potential water power, practically all developed, much of it in the Alpine regions of southern Baden and Bavaria and along the upper reaches and tributaries of the Rhine and the Danube. Its water power represents but a small portion of the total industrial power in use.

Austria, with about 1.7 mill. hp. of potential water power, has about 700,000 hp. developed.

In the *Balkan States*, Yugoslavia, Rumania, and Bulgaria all have considerable potential water power, totaling nearly 6 mill. hp. for these three states, and with about half of it in Yugoslavia. But little water power is developed as yet—about 350,000 hp. all told.

Greece has about 250,000 hp. of potential water power (capable of some increase by storage), with practically none developed. While this seems a small amount of power, considering the small area of this country (about 42,000 sq. miles), the amount per square mile—about 6 hp.—compares well with some of the other countries of Europe.

Italy has nearly 4 mill. hp. of potential water power. Recent progress in development has been rapid and nearly 5 mill. hp. of turbines are now in use. Most of the power is in the Alps and Apennines in northern Italy, where rivers of great fall, heading in regions of glaciers and perpetual snow, and with abundant mountain lakes for storage, result in some of the best water power sites in Europe.

Switzerland is remarkably endowed with water power for its size, with about 2.5 mill. hp. of potential power (minimum as estimated in 1925) and about 2.3 mill. hp. developed, or about 156 and 144 hp. per square mile of potential and developed water power, respectively—the leading country of the world in this respect. The potential power, with regulation was estimated (1925) at 8 mill. hp. This large amount of water power is due both to the great fall of its streams and well-sustained flow, due both to good rainfall and glacial and lake storage in the mountains. About two-thirds of the developed water power is for municipal and general use and about one-fourth is used in electrochemical industries. There are an unusually large number of small plants—some 6000 below 20 hp. in capacity and averaging only about 6 hp. each. Several combinations have been advantageously made of plants of large power but no storage with those of high head and storage. An impulse-wheel plant at the Dixence project utilizes the highest head at present developed—of 5715 ft.

France.—The potential water power of France at minimum flow is about 5.4 mill. hp., of which about 2.3 mill. hp. is developed. Most of the power is found along the eastern and southern borders, in the mountainous districts, the Rhone itself having over 800,000 potential

horse power at low water. In the 5-year period prior to 1921 over 800,000 hp. of water power was developed in France, of which approximately two-thirds is devoted to electrochemical and metallurgical work.

Spain has a higher average altitude than any other European country and contains many power sites but with rather small rainfall except on the northeastern coast. The largest plants are on the Ebro River and its tributaries, from which power is transmitted to Barcelona, Valencia, and Madrid. About 4 mill. hp. is available potentially, with about 1 mill. hp. developed.

Portugal is relatively low in elevation, although with a generally good rainfall, so that its potential water power is relatively small.

Great Britain.—The British Isles are, on the whole, at relatively low elevation but well supplied with rainfall. The largest potential water powers are in the Scottish Highlands, where at Kinlochleven there is a 30,000-hp. plant for the production of aluminum, supplemented by an additional 72,000 hp. at Ft. William, these figures representing, however, 12-hr. power. The total potential power of the British Isles is about 850,000 hp., with about 400,000 hp. developed. This includes the Shannon River development in Ireland, completed in 1929, where 100,000 hp. is developed under a head of 100 ft.

Water Power of Asia.—Asia is the largest continent, with practically one-third the land area of the globe. The chief feature of its drainage system is the great plateau between India and China, the highest plateau in the world, which ranges in altitude from 10,000 to 27,000 ft. On this plateau the surface slopes northward to the borders of Siberia, but even there has an altitude of 2000 ft. This high plateau is the source of all the large rivers of Asia. The inland drainage system of Asia has an area greater than that of the United States and extends into Europe, including the Volga and Ural rivers, whose basins constitute a large part of European Russia. This drainage basin has few fresh-water lakes, Baikal and Balkash being the only large ones. The larger part of the continent receives comparatively little precipitation, and only India, Indo-China, and a strip along the coast of China and Japan have a plentiful supply. The rivers flowing northward, therefore, do not attain large size until they reach their lower levels, and their potential power is small. In the coastal well-watered sections, which are the most thickly inhabited, little water power has been developed, except in Japan, although progress is gradually being made in this respect.

The *central and northern portions of Asia*, including the drainage area tributary to the Arctic Ocean, the Caspian Sea, the eastern Mediterranean and Dead Sea, the Black Sea and the Great Desert of Gobi in central Asia, include vast areas in which, due chiefly to lack of rainfall, the potential water power is small and there is practically no water power development.

Arabia and Persia also have practically no water power. The Tigris and Euphrates rivers afford some water power, although these streams are more important for irrigation purposes.

India contains approximately 27 mill. hp. of potential water power, with only about 400,000 hp. developed. The Indus River is of some importance, with a few developments already made. The western Ghats, a range of mountains along the west coast of the peninsula of India from Bombay southward, afford some good water power sites, with some opportunity for storage. In this district the rainfall from the middle of June to the middle of September averages about 175 in. Three plants of nearly 300,000-hp. capacity furnish power, which is transmitted about 80 miles to Bombay—almost entirely an industrial load. Over 500,000 hp. of potential power is available in these mountains. In eastern India the Salwin River and the Ganges afford large potential power, but little as yet developed. Abundant water power is also available on the island of Ceylon, due particularly to high rainfall and storage facilities.

In *China* a total of 20 mill. potential horse power of water power is estimated as available, with less than 2000 hp. developed. The Yangtze River, one of the two main rivers of China and a great artery of transportation, furnishes many opportunities for power development on its course, as well as on its tributaries, with a total potential power of upwards of 10 mill. hp. in an area which is densely populated, but with practically no water power development at present.

In *Japan*, due to heavy rainfall and mountainous areas, relatively large water power resources are found—6 mill. hp. of potential power at low water, with about 3.5 mill. hp. developed. Tokio now receives a large amount of power from the upper Fuji and Kinu rivers, as well as from a 60,000-hp. plant on Nippashi River from which power is transmitted about 145 miles. Other large plants have been built within recent years, the capacity of developed power being practically tripled during the last decade.

As will be noted from the table on page 30, Japan ranks high in potential water power per square mile of area and third in developed water power per square mile.

Water Power of Africa.—Africa, as previously noted, is a great elevated plateau which drops abruptly to the sea near the coast, resulting in great concentrations of water power due to the large size that the rivers attain in their lower courses where the falls occur. In northern Africa, however, the rainfall is less than 10 in. per year and potential water power small. Rainfall is also small in southern Africa, but in tropical Africa, comprising a strip along the Gulf of Guinea, the entire basin of the Congo, and the headwaters of the Nile, rainfall is abundant. Throughout this

region power sites are numerous, and on the lower Congo there are the greatest concentrations of potential water power in the world.

Liberia, on the west coast, has about 4 mill. potential horse power of water, which for the 40,000 sq. miles of its area is about 100 hp. per square mile—a very large unit amount as compared with other countries. *Sierra Leone* also has relatively large water power resources. No water power is developed, however, in this vicinity.

In *Nigeria* and the *British Kamerun* is about 9 mill. potential horse power of water power on the Niger River and its tributaries, an important power stream entirely undeveloped as yet.

In the *Belgian Congo* is about 90 mill. hp. and in the *French Congo* about 35 mill. hp. of potential water power. The Congo River drainage basin includes about 1.5 mill. sq. miles, nearly all with abundant rainfall. Its tributaries, themselves large rivers, in general flow from their sources to the central basin of the Congo in a succession of rapids, falls, or gorges, where conditions favor the development of power.

Stanley Falls, on the middle Congo, consist of seven cataracts having a total fall of about 200 ft. in 60 miles. From 10 to 15 mill. hp. may perhaps be developed in this locality, although the engineering problems involved would be very difficult.

On the lower Congo, below Stanley Pool, there are 18 falls and rapids having a total drop of 500 ft. in less than 90 miles and just above tide-water 10 more falls and rapids with a total drop of 300 ft. in less than 60 miles. In these two stretches of river more than 100 mill. hp. could be developed, although the difficulties of construction would be very great even if a market were available for this enormous quantity of power.

The Congo affords greater water power resources than any other river system in the world and more than all the other rivers in Africa combined. In fact, more than one-fourth of the potential water power of the world is in this one river basin, entirely undeveloped as yet, although some activity has prevailed in the last few years in respect to possible water power developments for railway operation and mining.

The Zambesi River in Rhodesia has some large potential power sites, notably at Victoria Falls, where the river falls abruptly about 350 ft., and in a distance of 50 miles there is an additional fall of about 1100 ft. The potential power of the falls and rapids has been estimated at about 750,000 hp. at low water.

The Shire River, the outlet of Lake Nyasa, an important tributary of the Zambesi, has large potential power, with a series of cataracts some 50 miles long in which the river makes the greater part of its descent in 1600 ft. from Lake Nyasa to sea level.

In *Egypt*, in a stretch of about 1800 miles between Khartoum and the Mediterranean, the Nile River has an aggregate fall of about 1300 ft., including six concentrations sufficiently great to be called cataracts.

The Assuan dam was built at the first cataract primarily to divert water for irrigation, but incidentally it affords a site, as yet undeveloped, capable of yielding 150,000 hp. for 5 months in the year. The cataracts of the Nile will probably not be developed for power except in connection with improvements in the river for navigation or irrigation.

Water Power of Oceanica.—*New Zealand* has about 2.5 mill. potential horse power of water, with about 160,000 hp. developed. It has abundant rainfall, with high mountainous areas and lakes available for storage. The government of New Zealand not only controls unutilized power sites but, together with municipal agencies, controls 90 per cent of the developed power. Since 1918 a complete scheme for supplying power to both the north and south islands has been adopted and is in process of execution.

Tasmania—the so-called Switzerland of the South—is said to be the most mountainous island of the globe. It has plentiful rainfall and numerous power sites. About 700,000 potential horse power is available, with about 10 per cent of this developed.

Australia, although nearly as large as the United States in area, has small water power resources, due chiefly to generally deficient rainfall. The only potential water power is in the mountains that border the east coast and would be rather high in cost of development, so that no plants have been built as yet. About 600,000 potential horse power of water is available.

New Guinea, the second largest island in the world, is mountainous and has a rainfall of more than 80 in. a year and abundant water power resources, estimated at at least 5 mill. hp. at low water, entirely undeveloped as yet.

Hawaii.—The Hawaiian Islands are mountainous and the rainfall is high but rather unevenly distributed, varying all the way from a few inches to a maximum of 600 in. annually in some localities. The available potential water power is about 100,000 hp., with about 30,000 hp. developed.

Comparative Water Power Resources.—The amount of water power, both potential and developed, per square mile of area, has been given by continents (page 5) and for certain individual countries where noteworthy. Another useful basis for comparison is the power available per thousand of population.

In Table 15 are given both of these unit factors for various countries, including in general those of greater importance or where water power resources are unusually large, based upon the 1930 estimates of the U. S. Geological Survey.

As will be noted, for developed water power, based upon population, Norway distinctly leads, with 715 hp. per thousand of population; and Canada closely second, with 708 hp. California alone, however, quite

comparable with Norway in size and population, has 435 hp. per thousand people. In fact, the three Pacific states average 475 hp. per thousand people, indicating the relatively high use of water power in this district.

TABLE 15.—COMPARATIVE WATER POWER RESOURCES

Country or subdivision	Population, thousands	Area, thousand square miles	Developed water power			Potential water power, minimum		
			Thousand horse power	Horse power per thousand population	Horse power per square mile	Thousand horse power	Horse power per thousand population	Horse power per square mile
Norway.....	2,650	125	1,900	715	15	9,500	3,580	76
Canada.....	9,934	3,730	7,045	708	1.9	18,000	1,810	4.8
Switzerland.....	4,018	16.0	2,300	573	144	2,500	620	156
Sweden.....	6,120	173	1,675	273	10	5,000	815	29
United States.....	123,000	3,027	15,818	129	5.2	38,000	306	12
France.....	40,746	213	2,300	57	11	5,400	133	25
Italy.....	41,168	118	4,840	117	41	3,800	61	32
Japan.....	62,938	149	3,500	56	24	6,000	96	40
Germany.....	67,812	183	2,000	30	11	2,000	30	11
Liberia.....	1,500	40	0	0	0	4,000	2,670	100
New Guinea.....	425	70	0	0	0	7,500	17,600	107
Europe.....	550,000	3,879	18,400	33	4.8	56,000	102	14.4
Africa.....	150,000	11,515	33	0	0.0	190,000	1,260	16.4
WORLD.....	2,000,000	52,373	46,000	23	0.9	447,000	223	8.5
New York.....	12,588	49.2	1,900	151	39	4,010	318	82
California.....	5,677	158	2,447	435	15	4,603	815	29
Washington.....	1,564	69.1	1,053	650	15	4,970	3,180	72
Oregon.....	954	96.7	364	370	3.7	3,665	3,840	40
New England.....	8,165	66.5	1,946	238	29	998	115	15
Pacific states.....	8,194	325	3,772	461	12	13,238	1,620	41

As regards developed water power as compared to area, Switzerland is well ahead, with 144 hp. per square mile, due to unusual concentration of power resources in a relatively small area. Several of the states or subdivisions of this country, however, taken individually show relatively high developed horse power per square mile, as, for illustration, New York with 39 hp. and New England with 29 hp.

Relative potential water power resources are best shown by the horse power available per square mile of area, although also shown in the table on the basis of population. Here again (excepting Liberia and New Guinea) Switzerland leads, with 156 hp. per square mile, followed by New York with 82 hp. and Norway with 76 hp. Liberia, as previously noted, has the remarkably high figure of 100 hp. per square mile, according to the 1921 estimates of the U. S. Geological Survey, which, however, are only approximate for this part of the world and for a region where no power is as yet developed. The island of New Guinea also appears to have relatively high potential water power resources.

It must be kept in mind that the potential water power as used in the foregoing table is the minimum power or that available at low water, and actual development when made would be to a materially larger amount, at least double the minimum power. The relative power of the different countries and states would, however, be similar and the comparative results and conclusions similar.

WATER POWER DEVELOPMENT IN THE UNITED STATES

Many of the older power developments of the country, aside from those in the West, have been made under laws or statutes similar to the so-called Mill Act in New England and New York, whereby a power site owned by an individual or corporation may be developed and, if necessary, flowage made of the land or other undeveloped water privileges upstream. Under this act, if riparian owners affected by a water power development refuse to sell their land or flowage rights at a reasonable amount, these may be taken and compensation fixed by court action. Such procedure could not, however, include the flowage of existing water power developments, as fundamentally such laws as the Mill Act were intended to foster and encourage the development of water power for industrial purposes.

In the West much of the available water power, including development of storage projects for its benefit, is located on the public or forest lands of the United States. For many years no fixed policy in respect to permits for its development prevailed and the necessary procedure requiring congressional action, greatly hindered such development.

President T. Roosevelt focused public interest on this matter by two messages to Congress in 1908 and 1909 vetoing acts conferring franchises for the development of water power in the public lands on the grounds that adequate provision for the protection of the interests of the general public had not been incorporated therein, stating that no rights involving water power should be granted to any corporation in perpetuity, but only for a length of time sufficient to allow them to conduct their business profitably, and that a reasonable charge should be made for valuable rights and privileges obtained from the national government.

During 1908 and 1909 extensive withdrawals of public lands of importance in respect to power development were made by the Department of the Interior and the Forest Service, which were in the nature of temporary power site withdrawals, awaiting action by Congress on this matter. Finally comprehensive action was taken by Congress by the passage of the Federal Water Power Act, approved June 10, 1920, and amended Mar. 3, 1921, to exclude all national parks and national monuments from its provisions. This Federal Water Power Act promises to be of the greatest importance in the development of the water powers of the West.

Interruptions in power service during and subsequent to the period of the World War, owing to shortage of fuel on account of both labor and transportation difficulties, have also called attention sharply to the need for more comprehensive water power development. One of the results of power shortage and interruptions in service has been the suggestion for the formation of "superpower zones." This was first made for the district including portions of New England, New York, and the middle Atlantic states, with a view to determining the future power needs of the district and methods for providing for these more efficiently and with more certainty.

The Federal Power Commission and Superpower are both of importance in the development of the water power of the country and warrant further consideration.

Federal Power Commission.—The Federal Power Commission, whose purpose, briefly, is "to provide for the improvement of navigation, the development of water power and the use of the lands of the United States in relation thereto," consists of a commission of five which performs its work in cooperation with personnel of the Departments of War, Interior, and Agriculture.

The commission, in cooperation with the other executive departments and with state agencies, is empowered to investigate the cost of water power development, its availability for market, and its fair value in any region to be developed.

Licenses are to be issued to citizens, corporations, municipalities, or states to construct, operate, and maintain projects for the development and improvement of navigation and for the development, transmission, and utilization of power (1) from or in any of the navigable waters of the United States and (2) upon any part of the public lands and reservations, and (3) to utilize the surplus water or power from any government dam.

Navigable waters means those parts of streams or other bodies of water over which Congress has jurisdiction under its authority to regulate commerce with foreign nations and among the several states, and which either in their natural or improved conditions, notwithstanding interruptions between the navigable part of such streams or waters by falls, shallows, or rapids compelling land carriage, are used or suitable for use for the transportation of persons or property in interstate or foreign commerce, including therein all such interrupting falls, shallows, or rapids, together with such other parts of streams as shall have been authorized by Congress for improvement of the United States or shall have been recommended to Congress for such improvement after investigation under its authority.

Licenses under the act are to be issued for a period not exceeding 50 years, giving preference as far as practicable to states and municipalities

in such applications. Annual license charges, reasonable in amount, are to be fixed by the commission, to cover cost of administration of the act and as recompense for the use of national lands or other property. Where a licensee is benefited by construction work of another licensee or storage reservoir or headwater improvement of the United States, the licensee so benefited shall reimburse the owner of such reservoir for such part of the annual fixed charges as the commission may deem equitable. If the licensee cannot acquire by contract or pledges the right to use or damage the property of others necessary for a project under the act, this may be done by exercising the right of eminent domain in the United States District Court.

The full text of the Federal Water Power Act is given in the *Third Annual Report* of the commission, Appendix A, pages 33 to 47. Rules and regulations prepared by the commission were issued in pamphlet form in 1921.

In the 11 years ended June 30, 1931, during which the act has been in effect, there have been filed with the commission 1168 applications relative to power developments, including transmission lines. Eliminating duplications, the active applications for power projects will involve, when built, an aggregate estimated installation of about 22 mill. hp.

Authorized licenses represent an installed first capacity of about 2,897,000 hp. and an estimated possible ultimate installation of about 7,406,000 hp. The former figure is about 20 per cent of the total hydroelectric generating capacity of the country, and 26 out of 51 states and territories are represented in its distribution—mostly in the West. Relatively little of this power is upon navigable streams outside of the federal domain.

The extent to which the commission has authority over rivers outside of the public and forest lands and Indian reservations is not clearly defined by the Federal Water Power Act. Such a river as the Connecticut, for illustration, is an interstate stream and is navigable in its lower portions. Theoretically a power or storage development on an upper tributary of the Connecticut might be said to affect the navigability of the stream and, hence, require action by the commission. Practically it would have no appreciable effect, and in any event any slight effect would certainly not be detrimental to navigation.

The commission has handled this phase of the matter by making a ruling, when requested in various specific cases, usually that there is no effect on navigation. There is, however, some uncertainty as to the necessity for applications in such cases and as to possible change in such rulings by future commissions, and it appears desirable definitely to limit in the act itself jurisdiction to the portions of rivers commercially navigable *in fact*. The jurisdiction of the commission was particularly intended to cover water power development in the various public lands

and at government dams on navigable streams and if extended farther, especially in an indefinite manner, is likely to be a hindrance rather than a help in encouraging the investment of capital in power projects.

Superpower.—The superpower investigation, reprinted as *Professional Paper* 123 of the U. S. Geological Survey and entitled, "A Superpower System for the Region between Boston and Washington," was made during 1920–1921 under the direction of the Geological Survey.¹

It included a survey and report upon:

. . . power production and distribution in the United States, including the study of methods for the further utilization of water power, and the special investigation of the possible economy of fuel, labor, and materials resulting from the use in the Boston-Washington industrial region of a comprehensive system for the generation and distribution of electricity of transportation lines and industries.

The superpower zone as considered in 1921 included portions of the three northern New England states, New York, Pennsylvania, Delaware, and Maryland, and all of the states of Massachusetts, Rhode Island, Connecticut, and New Jersey. Within this zone are concentrated about one-fourth of the population of the United States, as well as 315 electric utilities, 18 railroads, and 96,000 industrial plants.

The annual power requirement of this zone in 1930 was estimated to be about 31 bill. kw.-hr., of which approximately one-fifth could be supplied from water power. The system recommended comprehended large steam plants at tidewater and on inland waters, as well as the utilization of such hydroelectric power as could be economically obtained from rivers in or adjacent to the zone. The electric power so generated would be coordinated through a system of interconnected transmission lines of high voltage—220,000 and 110,000 volts.

It was estimated that the combined capital investment necessary for electric utilities and industries as of 1930 would be about \$1,294,000,000, which would net annually, above fixed charges, about \$429,000,000, or 33 per cent of the investment.

Of the 36,000 miles of railroad considered as single tract, it was concluded that about 19,000 miles could be profitably electrified to yield by 1930 an annual saving of \$81,000,000, as compared with 1920 steam operation. A capital expenditure of about \$570,000,000 would be required, and there would be an average return upon the investment, therefore, of about 14 per cent.

The coal that would be saved annually under the superpower system was estimated as follows:

¹ See also MURRAY, W. S., "Superpower—Its Genesis and Future," 1925.

	Million Short Tons
Electric utilities.....	19
Railroads.....	10
Manufacturing industries.....	21
Total.....	50

Many of the economies incident to superpower operation would be effected through the interconnection of existing plants and systems, which economies should be increased as new power plants and interconnections are added.

The procedure as suggested included the construction of large steam electric plants in the mine regions of Pennsylvania, and also near Boston and New Haven, and the construction of hydroelectric plants on the Delaware, Susquehanna, Hudson, and Potomac rivers.

Report of Northeastern Superpower Committee.—To stimulate interest and progress further in the superpower idea, the Northeastern Superpower Committee, under the leadership of Secretary Hoover, made further investigation and reported in July, 1924, giving attention to a somewhat larger superpower zone, including all of the states of Pennsylvania, New York, and West Virginia, and a portion of Ohio, and at the same time considering the use of power from the St. Lawrence River.

This committee found that in 1922 about 21 mill. kw.-hr. of power was used in the district, of which about 25 per cent was water power, and in addition nearly 14 bill. kw.-hr. yearly of mechanical power was used in industries, not including railroad transportation, which could be partially replaced by electric power. They found that the growth in electrical power requirements had practically compounded for the previous few years at the rate of 10 per cent per annum and estimated the power needed in 1930 (not including that for possible railway electrification) at about 31 bill. kw.-hr. Railroad electrification, if comprehensively carried out, would require something like 43 bill. kw.-hr. annually of power.

The economies possible with effective interconnection of utility systems include reduction of reserve equipment, better average load factor, more security in power supply, the advantageous use of water power, especially of secondary water power, and power available more quickly to meet growing demands.

The committee recommended the extension of interconnection between different power systems, the building of large centralized steam-electric plants, and the development of the large hydroelectric projects in the zone. Their studies indicated, however, that power generated in large steam plants at the mines could not be transmitted to distances greater than 200 miles and compete with power locally generated. Furthermore, the location of large steam plants is practically limited, due to

necessary water supply for condenser purposes, to the seaboard, Great Lakes, Ohio River and its tributaries, and the Susquehanna River.

Progress in Interconnection of Transmission Systems.—The 1921 and 1924 Superpower Reports attracted wide attention and helped to show the advantages of interconnection of transmission lines. Progress in this direction has been rapid since that time and interconnection of transmission-line systems has been practically accomplished in most of the country. The east, south, and central states form practically one large interconnected system, while the Pacific coast and adjacent states form another such system.

Future Water Power Development in the United States.—The essential features of "superpower," *viz.*, interconnection of transmission systems and the linking together of water power and large economical steam stations, are already practically accomplished. While the cost of steam power has been lowered by larger and more economical plants, the advantages of the combined use of water and steam power have been demonstrated. While the business depression of the last few years has temporarily lessened power demands, there should be in the future an opportunity for much water power development—especially of projects including storage, with its attendant advantages.

CHAPTER II

HYDROLOGY

RELATION OF WATER POWER TO HYDROLOGY

The basic elements of potential water power are (1) river or stream flow and (2) an available head or fall through which the stream flow may be utilized in the development of power. The determination of the head available at a water power site is usually a simple problem of surveying which can be worked out at any convenient time. The determination of the amount of stream flow and its variation at a power site is often a problem requiring much study, particularly where adequate flow measurements of the river in question are not available.

Hydrology in its broad sense has to do with the occurrence and distribution of water over and within the earth's surface and the study of the accompanying basic natural laws and phenomena. The estimation of available stream flow at a power site requires, therefore, a study in hydrology, and, where actual flow data are incomplete or unsatisfactory, often involves the consideration of the various factors affecting stream flow, such as precipitation, evaporation or water losses, percolation, etc., all of which are phenomena of hydrology.

Hydrology is a relatively new science, with few books as yet giving it special attention.¹ It is based upon meteorology, geology, agricultural physics, chemistry, and botany, as well as on a constantly increasing amount of data now being obtained by observation and measurement.

It must be kept in mind that deductions from such studies must be commonly limited to slight factors of safety, as compared, for example, with the design of ordinary structures where a factor of safety of 4 or 5 is common. Any such wide margin in water power estimates or flood protection, drainage, or irrigation studies—all fundamentally based upon hydrology—is usually out of the question from the point of view of economy or cost.

The study of hydrology with particular reference to stream flow is, therefore, fundamental in water power engineering, and the proper estimation of the available flow at a power site is a phase of the investigations worthy of the most careful attention and requiring the best judgment of the engineer.

¹ Mead's "Hydrology," and Meyer's "Elements of Hydrology," are good reference works for the student.

PRECIPITATION

Importance of Precipitation Data.—Precipitation (the general term used to include rainfall, snow, hail, etc.) is the first stage of the continual cycle of water in its round from atmosphere to earth and return by evaporation from land and water surface. The portion of this cycle in which the water is flowing down the rivers toward the oceans is, of course, that of direct importance in respect to water power, but, owing to lack of definite data of stream flow, it is frequently necessary to study and use the factors in the other portions of the cycle, *viz.*, precipitation and evaporation, and to determine or check indirectly the water available for stream flow.

Observations of stream flow are quite limited, in both time and geographical extent, although this subject is receiving general attention by the Water Resources Branch of the U. S. Geological Survey so that flow records of 15 to 20 years' duration are now available for many rivers. Observations of precipitation have frequently extended over long periods of time, and the points of observation are geographically widely distributed. These precipitation records are, therefore, of service in helping the engineer to extend his predictions made on the basis of short time or meager stream-flow data and to estimate approximately the probable variation in the yield of streams over periods of considerable length. It is, therefore, important that the engineer should understand the relations that exist between precipitation and stream flow, and the modifications of these relations by other physical factors. He should also know where precipitation records can be obtained and how to use them.

Causes of Precipitation.—A complete discussion of the causes of precipitation and its variation in distribution and amount would be very extended, as well as imperfect in many details, since the factors involved are conditions of world-wide extent and are not fully understood. Some of the simpler principles underlying the general causes of precipitation and its distribution with particular reference to the United States, however, may well be considered.

All air contains a certain amount of moisture, ranging from the amount sufficient to saturate it, to a very small proportion of that quantity. The point of saturation is called the dew point, or, in other words, the dew point may be defined as the temperature to which the air must be cooled to cause condensation of the vapor therein. Warm air can hold a much larger quantity of moisture than cold air; in fact, as will be noted from Table 16, the amount of water in saturated water vapor practically doubles for each increase in temperature of 20° F. for ordinary air temperatures.

Whenever air is cooled below the dew point, either precipitation occurs or particles of water are condensed as clouds and float away. The

TABLE 16.—ELASTIC PRESSURE OF SATURATED WATER VAPOR AT DIFFERENT TEMPERATURES
(U. S. Weather Bureau Bull. 235)

Air temperature, degrees Fahrenheit	Vapor pressure, inches of mercury	Air temperature, degrees Fahrenheit	Vapor pressure, inches of mercury
0	0.0383	50	0.360
1	0.0403	51	0.373
2	0.0423	52	0.387
3	0.0444	53	0.402
4	0.0467	54	0.417
5	0.0491	55	0.432
6	0.0515	56	0.448
7	0.0542	57	0.465
8	0.0570	58	0.482
9	0.0600	59	0.499
10	0.0631	60	0.517
11	0.0665	61	0.536
12	0.0699	62	0.555
13	0.0735	63	0.575
14	0.0772	64	0.595
15	0.0810	65	0.616
16	0.0850	66	0.638
17	0.0891	67	0.661
18	0.0933	68	0.684
19	0.0979	69	0.707
20	0.103	70	0.732
21	0.108	71	0.757
22	0.113	72	0.783
23	0.118	73	0.810
24	0.124	74	0.838
25	0.130	75	0.866
26	0.136	76	0.896
27	0.143	77	0.926
28	0.150	78	0.957
29	0.157	79	0.989
30	0.164	80	1.026
31	0.172	81	1.062
32	0.180	82	1.091
33	0.187	83	1.127
34	0.195	84	1.163
35	0.203	85	1.201
36	0.211	86	1.241
37	0.219	87	1.281
38	0.228	88	1.322
39	0.237	89	1.364
40	0.247	90	1.408
41	0.256	91	1.453
42	0.266	92	1.499
43	0.277	93	1.546
44	0.287	94	1.595
45	0.298	95	1.645
46	0.310	96	1.699
47	0.322	97	1.749
48	0.334	98	1.803
49	0.347	99	1.859
		100	1.916

immediate cause of precipitation is, therefore, dynamic cooling due to the upward movement of moist air. According to Curtis,¹ this upward movement of air and consequent cooling accompanied by precipitation may be:

1. Orographic, due to hills or mountains forcing the air currents upward by contact.

2. Convective, due to substantially vertical circulation of warm moist air and, hence, a more or less continual restoration of moisture previously evaporated.

3. Cyclonic, due to prevailing storm movements (as in the United States) where, in addition to a horizontal movement of air currents away from sources of moisture, there is a vertical component, due in part to convection. With this type of precipitation the moisture evaporated from one region may be precipitated over another at some distance away.

An important cause of variation of precipitation during different portions of the year is the fundamental difference between land and sea as regards seasonal temperatures. The sea receives heat slowly and parts with it slowly; the land, on the contrary, is rapidly heated and parts with its heat as quickly. The ocean, therefore, has a fairly uniform temperature the year round, while the land is much colder in winter than in summer.

As, in general, the chief source of precipitation is water previously evaporated, it follows that proximity to the ocean and other large bodies of water tends toward a greater amount of local precipitation, modified largely, however, by the prevailing direction of winds and topography, as will be further noted.

It is evident, assuming a stable condition of water level of the ocean, that only that portion of the rainfall which runs back into the ocean as stream flow represents water evaporated from the ocean. The remainder comes from land evaporation. Thus, of the 29 in. of average yearly rainfall in the United States, probably about 12 in. flows back into the ocean and, hence, is evaporated from it, while the remaining 17 in. represents water evaporated from the land and continually reprecipitated. In the broad sense, therefore, the oceans are the true sources of our water power.

Winds are temperature effects, caused by the unequal heating of the earth's surface. The general tendencies in the western hemisphere are as follows: In the tropics there is a belt of calms; the daily air movement is vertical or convectional, with resulting heavy daily rainfall. Each side (north and south) of the tropical belt of calms is a region where air is moving toward the equator, deflected into a westerly direction, however, by the earth's rotation and resulting in the northeast trade winds

¹ U. S. Forest Service, *Bull.* 7, p. 187.

in the northern hemisphere and the southeast trade winds in the southern hemisphere. In the upper air the return or antitrade winds blow in the opposite direction.

At about 30° north and south latitude the air currents are vertically downward—the calms of Cancer and Capricorn—with low humidity and rainfall. Between latitude 30° and the poles, the most populated regions, the winds are “prevailing westerlies,” dominated by “lows” and “highs” resulting from the unequal heating of the land and water masses, which cross the continents at a normal rate of 600 to 700 miles per day. These are “cyclonic movements” and determine our weather.

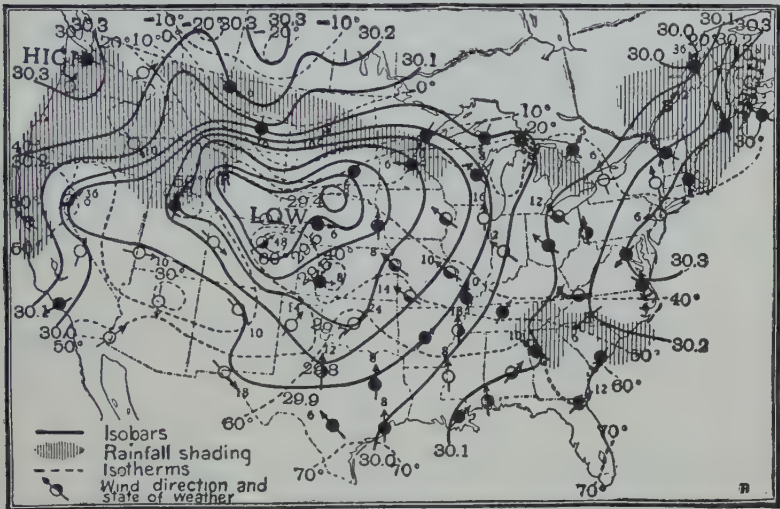


FIG. 3.—Daily weather maps of U. S. Weather Bureau. March 2, 1904, illustrating a low.

Storms in the United States.—A study of the daily weather reports and maps of the U. S. Weather Bureau and the tracing of the paths and phenomena of different storms will help one materially in understanding the variations in precipitation in different parts of the country and the general directions from which storms come.

These reports, based upon telegraphic information, usually at 8 A.M., in addition to local and district weather forecasts, show data of temperature, precipitation, barometric pressure, wind direction, etc. A storm movement or tendency is clearly indicated by a “low” or center of low barometric pressure, while a “high” area commonly means fair weather.

As will be noted from typical maps (see Figs. 3 and 4), air currents are in general flowing from high to low areas, although in the vicinity of the low area wind directions are counterclockwise, precipitation usually occurring somewhat in advance of the low center.

Two general classes of storms may be recognized, those first appearing at or near the Pacific coast and moving, in general, easterly, and those

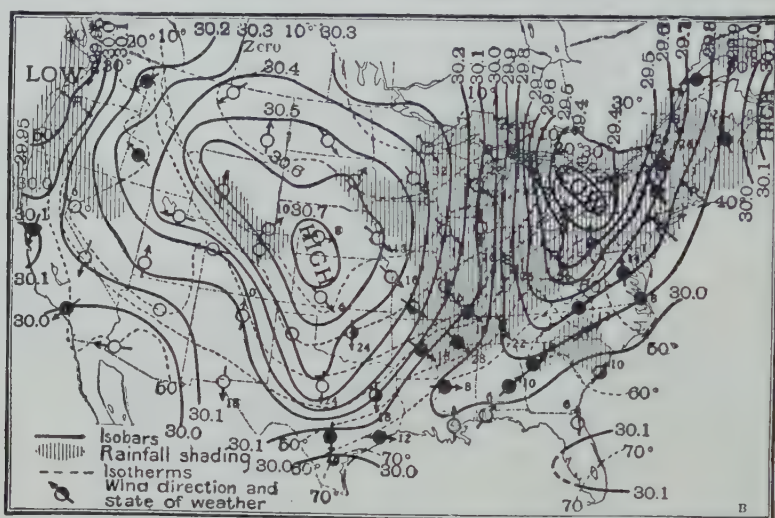


FIG. 4.—Daily weather maps of U. S. Weather Bureau. March 3, 1904, illustrating a *low* and the succeeding *high*.

originating in the South Atlantic, often appearing in or near the Gulf of Mexico and moving generally northeasterly.

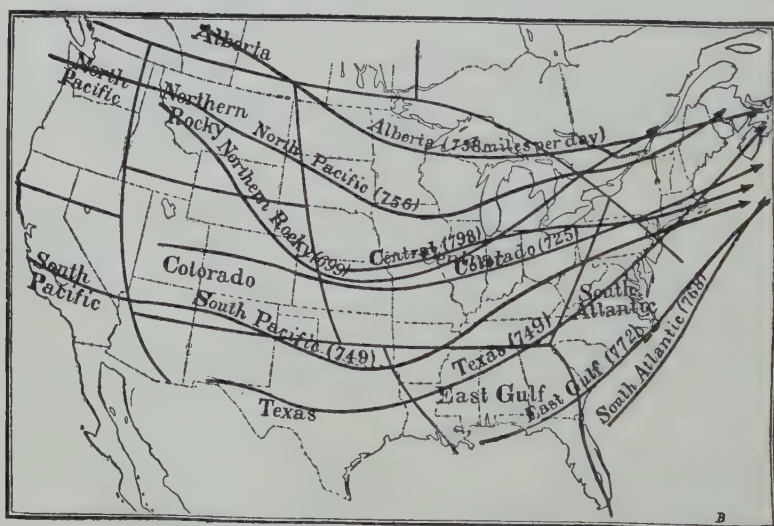


FIG. 5.—Storm paths in United States. Average path of lows, January.

Of course no two storms follow exactly the same path, and there are many combinations and modifications of the two general types assumed (see Figs. 5 and 6).

A well-defined storm movement with widely spread low area whose source is the warm moist currents of the Pacific may cause precipitation in the West and passing eastward draw toward its center and precipitate moisture which evaporated from the Atlantic as well as from the land and water area between the Atlantic coast and the storm center. Similarly, a storm passing northeasterly along or near the Atlantic coast may precipitate moisture evaporated from the interior of the country as well as that coming from the ocean.

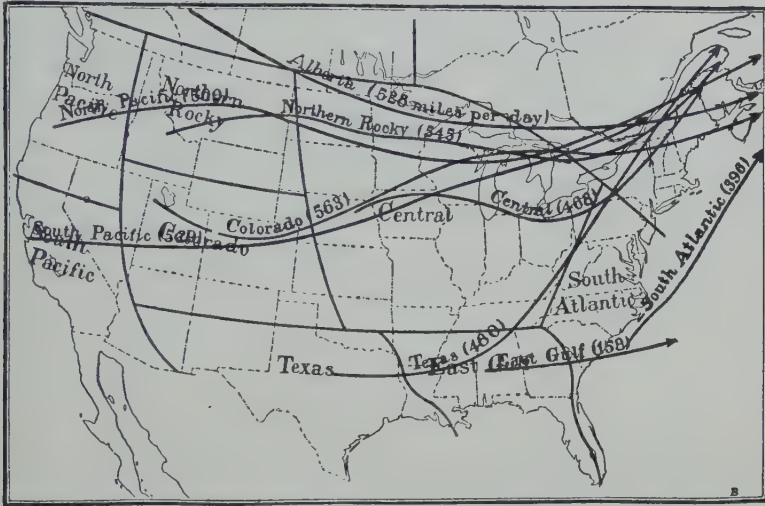


FIG. 6.—Storm paths in United States. Average path of lows, July.

Precipitation in the United States.—Keeping in mind the basic causes of precipitation as well as the general paths of storm movements, the following will be seen:

The prevailing winds from the Pacific Ocean are from the west and have the temperature of the ocean. If they encounter land having a temperature below the dew point, they are chilled below the point of saturation and some of the moisture is deposited in the form of rain or snow. If, on the other hand, the land is warmer than the air, the currents pass over it without any reduction in temperature and with little or no loss of moisture.

During the winter the north Pacific coast is colder than the sea, and consequently the precipitation is very large in that season of the year. In the summer the conditions are reversed, and, while the air currents from the Pacific Ocean contain as much moisture as they did in cold weather, they pass over the land with little loss from precipitation. On the south Pacific coast, the land is progressively warmer in winter and consequently receives less rain. In southern California there is little

rain even in winter, except upon the mountains. The Pacific coast, therefore, has well-defined wet and dry seasons corresponding to winter and summer in other parts of the country. In winter the country drains



Fig. 7.—Isohyetal map of United States.

the air currents of their moisture and they pass eastward as dry winds, while in summer these currents carry some of their moisture over the mountains and precipitate it upon the Rocky Mountains farther to

the east and upon the Great Plains. Consequently, in these two latter regions the greater part of the year's rainfall occurs in the warmer half of the year.

Going eastward into the Mississippi Valley the moist air currents, drawn northward from the Gulf of Mexico by southerly winds, are carried progressively into more northerly and colder climates, and part of their moisture is deposited in a similar manner. Along the Atlantic coast precipitation occurs in a similar manner and results largely from the moisture-laden winds coming from the Atlantic Ocean and in part from the Gulf of Mexico.

A map of the country on which is plotted isohyets of mean annual precipitation (see Fig. 7) shows clearly (1) the tendency toward greater precipitation near the coast, (2) high precipitation in the mountains near the coast, and (3) very low precipitation on the land side of the higher mountains (in the West), even though near the source of aqueous vapor, due to its previous removal by contact with the cold slopes lying toward the ocean.

The mean annual precipitation for the whole country is about 29 in. varying from a maximum of over 130 in. near the coast in Oregon to a few inches in southern Nevada. The amounts at a few typical stations are shown in Table 17.

Measurement of Precipitation.—The greater portion of data upon precipitation is obtained and the results published by the U. S. Weather Bureau. There are now several thousand regular so-called cooperative observer stations where daily observations of precipitation, temperature, and weather conditions are noted and over 100 regular stations from which telegraphic reports are made daily. There are also some records available which are kept by municipal and state authorities, as well as private corporations.

Rain Gages.—A rain gage for measuring precipitation consists of a collecting cylinder which exposes a circular surface for collecting the rainfall and a storage vessel in which the water is retained until measured. The standard rain gage of the U. S. Weather Bureau, in most common use, consists of a receiver, an overflow cylinder, and a measuring tube (see Fig. 8). In this rain gage the exposed receiver area is 8 in. in diameter and is connected by means of a reducing funnel with the measuring tube, which has an inside area of cross section of one-tenth of the area of the surface of the receiver. The measured depth of water in this tube is, therefore, ten times the depth of precipitation, and this is observed by means of a cedar measuring stick to the nearest hundredth of an inch of rainfall. When used as a snow gage, the funnel and measuring tube should be removed and the overflow cylinder used to catch the snow, which should be melted, poured into the measuring tube, and measured with the stick as with rainfall.

TABLE 17.—MEAN ANNUAL PRECIPITATION IN INCHES

Location	Elevation, feet	Precipitation, inches
Portland, Me.....	90	42.6
Concord, N. H.....	350	39.8
Mt. Washington, N. H.....	6293	81.7
Burlington, Vt.....	404	32.7
Boston, Mass.....	124	44.7
Hartford, Conn.....	159	44.3
Albany, N. Y.....	97	38.4
Philadelphia, Pa.....		40.9
Washington, D. C.....	75	40.8
Hatteras, N. C.....	11	60.2
Charleston, S. C.....	48	48.6
Atlanta, Ga.....	1218	49.5
Mobile, Ala.....	57	61.8
Memphis, Tenn.....	409	50.2
Cincinnati, Ohio.....	628	40.9
St. Louis, Mo.....	568	40.1
Detroit, Mich.....	730	32.1
Madison, Wis.....	974	31.2
Duluth, Minn.....	1133	29.8
New Orleans, La.....	51	55.6
Austin, Tex.....	593	33.0
Topeka, Kan.....	992	34.1
Yankton, S. D.....	1234	26.0
Great Falls, Mont.....	3350	14.8
Cheyenne, Wyo.....	6088	13.8
Boise, Ida.....	2770	13.3
Spokane, Wash.....	1943	17.9
Olympia, Wash.....		55.2
Clearwater, Wash.....		128.4
Pendleton, Ore.....	1272	14.1
Portland, Ore.....	57	42.5
Glenora, Ore.....	575	132.8
Sacramento, Calif.....	71	19.4
San Francisco, Calif.....	207	22.8
Los Angeles, Calif.....	293	15.9
Carson City, Nev.....	4660	11.0
Salt Lake City, Utah.....	4366	16.3
Denver, Colo.....	5272	14.0
Albuquerque, N. M.....	5200	7.5
Prescott, Ariz.....	5320	17.4
Yuma, Ariz.....	141	3.1
El Paso, Tex.....		9.3
Galveston, Tex.....	69	46.3

Recording Rain Gages.—At the more important Weather Bureau stations recording gages are used and a continuous record obtained. The common type is that known as the “tipping bucket” gage (see Figs. 9 and 10), where rain from the collector runs into a small bucket, divided into two parts and mounted on trunnions so placed that when one part of the bucket (holding an amount corresponding to $\frac{1}{100}$ in. of rainfall) is filled, it tips over, closing an electrical circuit and recording by a pen upon a clock-operated record sheet. Such gage records are of value in studying the intensity of rainfall, particularly during short periods of time, and are important as a basis for the design of storm-water sewers but are not ordinarily of significance in water power studies.

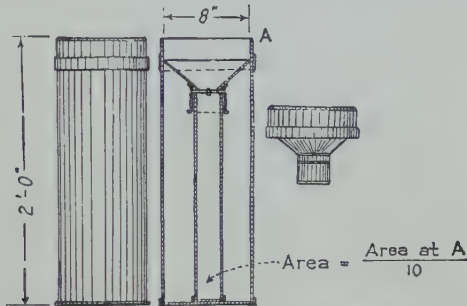


FIG. 8.—Standard rain gage—U. S. Weather Bureau.

Precipitation Records—Errors.—Errors in precipitation measurements are due chiefly to (1) poor location or exposure of the rain gage and (2) the incorrect collection and measurement of snowfall, although occasionally observational and (particularly in the case of old records) instrumental errors may be found.

On level ground remote from trees or buildings, a gage is likely to be subject to severe wind action unless shielded; on rolling ground or in a built-up district, local eddies in air currents may cause an incorrect catchment. The best location calls for the use of good judgment based upon experience.

The effect of wind upon the catch of rain gages is stated by Abbe, in part, as follows:¹

When the wind strikes an obstacle, the deflected currents on all sides of the obstacle move past the latter more rapidly; therefore, the open mouth of the rain gage has above it an invisible layer of air whose horizontal motion is more rapid than that of the wind a little distance higher up. Of the falling raindrops the larger ones may descend with rapidity sufficient to penetrate this swiftly moving layer, but the slower falling drops will be carried over to the leeward side of the gage, and failing to enter it, will miss being counted as rainfall, although they go on the ground near by. . . .

¹ “Forest Influences,” U. S. Weather Bureau, *Bull.* 7.

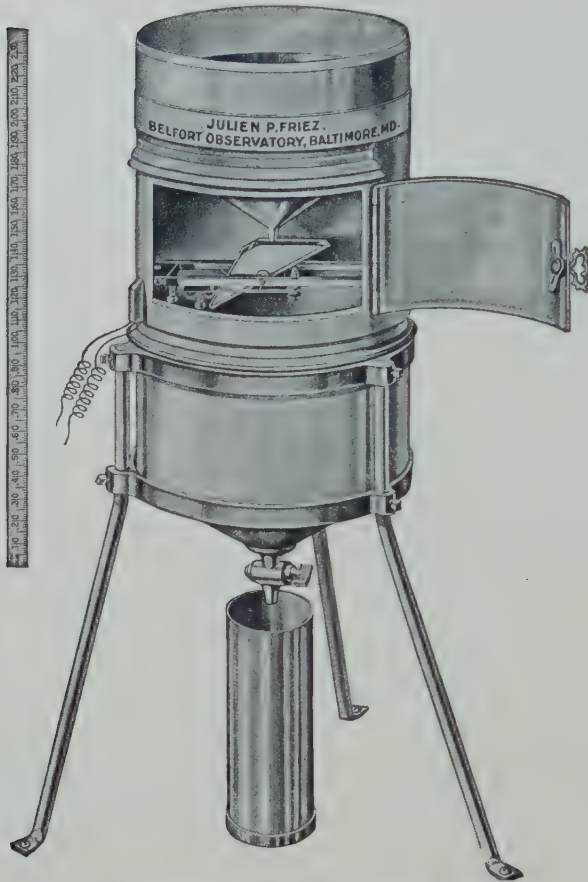


FIG. 9.—Friez tipping-bucket rain gage. (Courtesy of Julien P. Friez and Sons.)

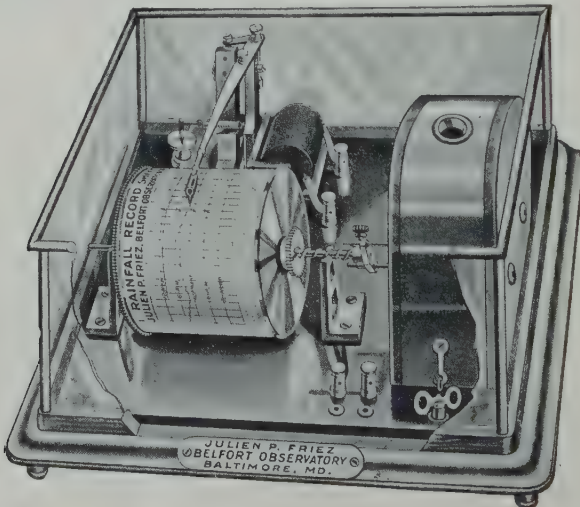


FIG. 10.—Recorder for tipping-bucket rain gage. (Courtesy of Julien P. Friez and Sons.)

The action of the wind in blowing the precipitation over to the leeward of the gage . . . is aggravated by the formation of whirls or eddies within the gage itself by reason of which light and dry snowflakes are even whirled out of the gage, after being once caught in it. Similar remarks apply to the rainfall on the top of a large square building with a flat or depressed roof; not only does the top as a whole receive less than an equal area at the ground, but the distribution of rainfall on the roof is such that the least rain falls on the windward portion and the most on the portion to leeward, while somewhere on the roof will be found a region whose average rainfall coincides with that on the ground. But the location of this region will vary with the direction and strength of the wind and the quality of the precipitation, so that we have but little assurance that any single rain gage on the roof will represent the rainfall on the ground. . . .

If properly protected from the wind, a gage placed at any altitude below the cloud level will unquestionably catch as much rain as one placed at the ground surface.

Where rain gages are located on the top of high buildings, as is often the case with the U. S. Weather Bureau city stations, unless shielded by parapet walls, the catchment is likely, therefore, to be too small by 5 to 10 per cent, or even more in some cases. This possibility of error should be kept in mind in making use of data at such stations.

As an illustration of this effect, in Table 18 are given precipitation data for three stations at Providence, R. I.

TABLE 18.—YEARLY PRECIPITATION AT PROVIDENCE, R. I. (1906-1920, INCLUSIVE)

Item	Station		
	U. S. Weather Bureau	Water-works reservoir	Water-works pumping station
Yearly precipitation, inches.	36.70	42.89	43.41
Ratio to precipitation at Weather Bureau.	1.00	1.17	1.18
Elevation, feet.	182	182	25

These stations are but a few miles apart. The Weather Bureau gage is located upon the top of a high office building, probably upwards of 150 ft. above street level and, as will be noted, shows a deficiency of about 17 per cent in rainfall, as compared with the other gages.

Where rain gages must be located above ground level or in very exposed positions, a protective shield may be used to minimize wind effect. The Nipher shield, invented by Prof. F. E. Nipher of St. Louis in 1878¹ is an effective type of shield, which, however, has been but little used in this country. Figure 11 shows a rain gage equipped with a shield of the Nipher type.

¹ NIPHER, F. E., "On the Determination of the True Rainfall by Elevated Gages," *Proc. Amer. Assoc. Adv. Sci.*, 1878, pp. 106-108.

For measurement of snowfall the interior measuring cylinder and funnel of the rain gage are removed and the snow caught in the 8-in. cylinder melted and then measured by transfer to the measuring cylinder. The results of such measurements are often faulty, however, as light snow is often blown out of the gage. Horton¹ compared the gage catch of snow with the true snowfall determined by a sample of undrifted snow on level ground, in a storm at Albany, N. Y., Dec. 26, 1913, finding

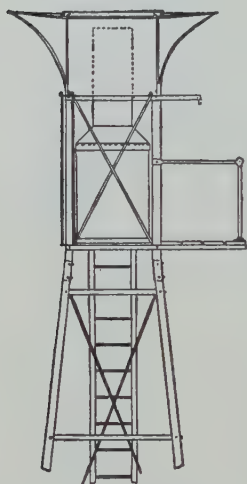


FIG. 11.—Nipher shield for rain gage.

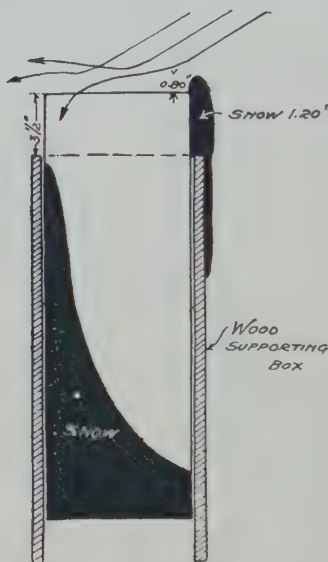


FIG. 12.—Action of snow on rain gage.

0.43 in. in the gage can, as compared with 1.41 in. on level ground. The action of the snow in this case is shown in Fig. 12.

Horton² made a series of comparisons of snow catch in the rain-gage can with that upon a snow board through the winters of 1919–1920 and 1922–1923, near Albany, N. Y., obtaining a correction factor to be applied to the gage-can measurements to obtain the true snowfall, expressed as water equivalent, of 1.16 as an average for the four winters, and varying from 1.12 to 1.20.

He suggests a tentative correction factor,

$$\text{Percentage increase} = 1.5(\phi - 35) + \frac{E}{500},$$

¹ HORTON, R. E., "The Measurement of Rainfall and Snow," *Jour. New Eng. Water Works Assoc.*, March, 1919, pp. 31–32.

² HORTON, R. E., "Determining the Mean Precipitation on a Drainage Basin," *Jour. New Eng. Water Works Assoc.*, March, 1924.

where ϕ is the latitude and E the elevation of the gage, to be applied to gage-can observations, November to April, inclusive, to obtain the estimated true winter precipitation.

Preferably, however, snowfall should be measured upon the ground or better on a wooden platform, in selected spots where the depth is normal, inverting the measuring tube of the rain gage, pushing it down through the snow and thus obtaining a prism of snow which should be melted and measured with the stick. In some of the older records a water equivalent of 0.1 was applied to depth of snow, which often led to serious error, as the actual water equivalent may vary from 0.25 to 0.05 or less.

The precipitation data of the U. S. Weather Bureau must, therefore, be generally considered as approximate, particularly for the earlier records, and there is little advantage gained in making studies of precipitation by any elaborate system of weighting of results for different stations in a drainage area. Data of precipitation as ordinarily observed are not usually good to more than one decimal place, although usually recorded to hundredths of an inch.

Range in Annual Precipitation.—Great variations take place in the annual precipitation of every locality, but apparently in no regular cycles, as far as can be told from the records available in this country, few of which exceed 100 years in length.

TABLE 19.—ANNUAL PRECIPITATION—VARIATION FOR SINGLE YEAR

Station	Years of record	Mean, inches	Minimum		Maximum	
			Per cent of mean	Year	Per cent of mean	Year
Boston, Mass.....	100	43.86	62	1822	154	1863
Amherst, Mass.....	78	44.15	69	1908	131	1888
Albany, N. Y.....	88	37.98	69	1913	150	1871
Newark, N. J.....	65	47.75	72	1856	145	1903
Washington, D. C.....	63	40.80	57	1848	150	1889
Augusta, Ga.....	63	46.94	41	1845	122	1874
New Orleans, La.....	74	55.63	56	1899	154	1875
Marietta, Ohio.....	90	42.20	62	1904	146	1858
Chicago, Ill.....	38	33.54	73	1901	132	1885
Leavenworth, Kan.....	73	34.80	42	1864	171	1858
North Platte, Neb.....	34	18.74	60	1894	160	1883
Denver, Col.....	37	14.05	60	1893	152	1891
Browawe, Nev.....	35	6.48	33	1905	230	1891
Astoria, Wash.....	55	75.35	66	1884	144	1879
San Francisco, Calif.....	51	22.83	41	1897-1898	170	1883-1884
Los Angeles, Calif.....	32	15.86	30	1897-1898	253	1883-1884
San Diego, Calif.....	59	9.54	32	1862-1863	290	1883-1884

In Table 19 is shown the relation of maximum and minimum annual precipitation to the mean for a number of selected stations. It will be noted that where the mean exceeds about 15 in. a minimum of about 40 per cent and a maximum of about 170 per cent of the mean may occur in some localities, although a range of from about 60 to 150 per cent of the mean is more common.

Under a total of 15 in. (in Nevada, California, etc.) the range is from about 30 to 250 per cent or more of the mean.

Table 19 shows the variation in precipitation that may be expected for a single year. Not infrequently years of abnormal precipitation (either high or low) may occur in succession, thus causing a greater effect upon the yield of streams than if coming singly. The following appears for certain stations in Table 19:

TABLE 20.—ANNUAL PRECIPITATION—VARIATION FOR PERIODS OF 1 TO 5 YEARS

Station	Per cent of mean annual precipitation							
	Minimum				Maximum			
	Single year	Successive years			Single year	Successive years		
		Two	Three	Five		Two	Three	Five
Boston, Mass.....	62	73	83	85	154	147	136	127
Washington, D. C.....	57	72	79	85	150	138	131	116
Leavenworth, Kan.....	42	61	66	77	171	142	124	115
San Francisco, Calif.....	41	56	72	74	170	146	141	118

Evidently annual precipitation may average from about 15 to 25 per cent either above or below the mean for a period as long as five successive years.

The student should study some reliable precipitation record in the above manner, extending the scope of the study to a longer period, plotting the results, and comparing with a curve of precipitation frequency, as explained below, obtained by taking values in order of magnitude instead of chronologically and using per cent of time as abscissas.

Frequency Curves of Precipitation.—On Fig. 13 is shown a frequency curve of yearly precipitation at Amherst, Mass., for the period 1836–1920, inclusive, or 85 years, with values arranged in order of magnitude, rather than the calendar order of occurrence. In this case the per cent of time during which any given amount of precipitation is equaled or exceeded is shown. Thus, a precipitation equal to, or greater than, 30.68 in. (the lowest observed) occurred 100 per cent of the time (or more strictly 99.415 per cent of the time), while 58.04 in. (the highest observed) may be considered as the 0 per cent value (or more strictly the 0.585 value).

Note that the mean or average of 44.17 in. is greater than the normal or median amount of 43.53 in., due to the preponderant effect of the higher values upon the average.

If desired, the curve may be made by plotting as ordinates the ratios to the mean precipitation instead of actual amounts.

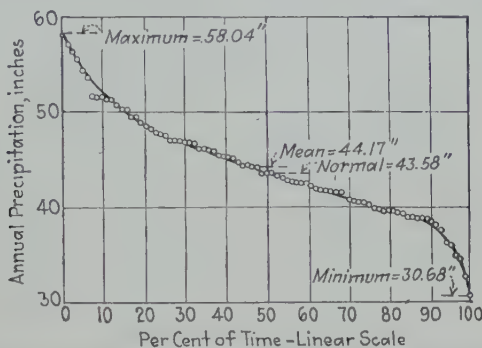


FIG. 13.—Frequency curve of precipitation at Amherst, Mass., 1836-1920. Linear scale.

To show more clearly the trend of the extreme values in such plots, Hazen has suggested the use of probability paper¹ where, instead of coordinates being plotted in linear dimensions, a scale of distances

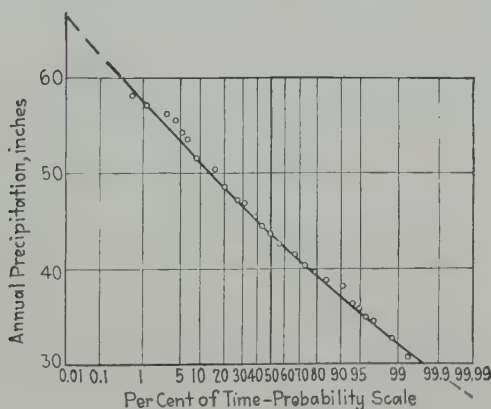


FIG. 14.—Frequency curve of precipitation at Amherst, Mass., 1836-1920. Probability scale.

varying in accordance with values of the probability integral, $y = ke^{-h^2x^2}$, are used, where x is the error and y its probability; and $k = \frac{h}{2\pi}$, h being a constant for a given series of observations. Such a plot is shown in Fig. 14 using the same data of precipitation for Amherst as in Fig. 13.

¹ *Trans. A.S.C.E.*, vol. 77, p. 1668, 1914. See also MERRIMAN and WIGGIN, "American Civil Engineers Handbook," pp. 78-79.

As will be noted, the whole curve is more nearly a straight line, and its extremes may be more readily extended to show the tendency toward values beyond the plotted limits of observation.

Probability paper for plotting is now available or the student may readily construct it from Table 21, which gives relative values of abscissas for percentages either side of the 50 per cent or median value.

TABLE 21.—RELATIVE DISTANCES OF LINES ON PROBABILITY PAPER FROM CENTRAL OR 50 PER CENT LINE

Line per cent	Relative distance	Line per cent	Relative distance
50	0.000	1.0	3.450
45	0.186	0.8	3.573
40	0.376	0.6	3.727
35	0.571	0.5	3.821
30	0.777	0.4	3.933
25	1.000	0.3	4.077
20	1.248	0.2	4.267
15	1.537	0.1	4.583
10	1.900	0.08	4.685
8	2.083	0.06	4.817
6	2.305	0.05	4.900
5	2.439	0.04	5.000
4	2.596	0.03	5.120
3	2.789	0.02	5.290
2	3.045	0.01	5.550

The following computations and conclusions may be made from Fig. 14:

TABLE 22.—PROBABILITIES OF PRECIPITATION AT AMHERST

Relative time of recurrence, per cent	Precipitation, inches		Frequency once in Years	Deviation from mean, per cent	
	+	—		+	—
25	47.7	40.0	4	7.9	9.4
10	51.0	37.0	10	15	16
5	53.5	35.2	20	21	20
1	57.6	32.0	100	30	27
0.1	62.5	28.6	1,000	41	35
0.01	66.5	26.4	10,000	50	40

The above conclusions are well established in the portions of the curve near the mean. Thus, every 4 years departures from the mean of 8 or 9 per cent in either direction may be expected; every 10 years departures of about 15 per cent, etc. The conclusions in the last three lines become

more and more uncertain, however, and serve to show nothing more than that over a very long period of time a departure of about +50 per cent and -40 per cent may be expected, which conclusions are, however, reasonable when compared with data in Table 19 on page 51.

Such a frequency curve for San Francisco, for example, would show much greater variations from the mean than that for Amherst. Hazen¹ has expressed this variation in the following manner, using data for Amherst as an illustration:

The "standard variation" = $\sqrt{\frac{\Sigma v^2}{n-1}} = \sqrt{\frac{2606.3}{85-1}} = 5.57$ where v is the variation from the mean and n the whole number of observations.

The "coefficient of variation" = $\frac{5.57}{44.17} = 0.126$, a quantity which reflects the variation in yearly precipitation in terms of the mean.

Hazen prepared a map of the United States showing values of this coefficient of variation. These vary from about 0.10 to 0.20 in the East to 0.20 to 0.50 in the West and, therefore, show that, on the average, relative variations in annual precipitation are about twice as great in many western states as in the East.

It must be kept clearly in mind, however, that such conclusions as may be drawn from frequency curves disclose nothing whatever as to calendar or actual order of occurrence of any given amount of precipitation. It cannot be foretold, for illustration, when the very dry or very wet year may occur. The longer the record, however, the more accurate becomes the definition of probable range in precipitation. This subject is further discussed under Rainfall Frequency, page 68.

Period Required to Determine Mean Precipitation.—As precipitation varies in an irregular manner, a determination of the mean annual amount requires a considerable number of years of observation.

Binnie² has stated that dependence can be placed on any good record of 35 years' duration to give a mean rainfall correct within 2 per cent of the truth. A 20-year record may be expected to show an error of a little over 3 per cent, while the probable extreme deviation from the mean for the shorter periods of 5, 10, and 15 years would be about 15, 8, and 5 per cent, respectively.

Professor Henry, of the U. S. Weather Bureau, has concluded that somewhat longer periods are required to give the above accuracy, and that about 40 years' observations are required to give results within 15 per cent of the true normal.

Many of the records of the U. S. Weather Bureau began about the year 1890, so that, generally speaking, 30 years' records are available at many points. It is probable that a 20-year record furnishes results

¹ *Eng. News*, Jan. 6, 1916.

² See *Proc. Inst. Civil Eng.*, vol. 109, p. 89, 1892.

with as high a degree of accuracy as can be utilized, considering the way such data can be applied in estimates of stream flow, and it is often possible to find at least one long-term record extending back for 50 years or more upon which long-time estimates or studies can be based.

The variation in yearly mean and the number of years required to establish a value approximately that of the long-time mean may be studied by either tabulation or plot, beginning at the latest year of record and computing the progressive mean backward year by year (see Fig. 15).¹

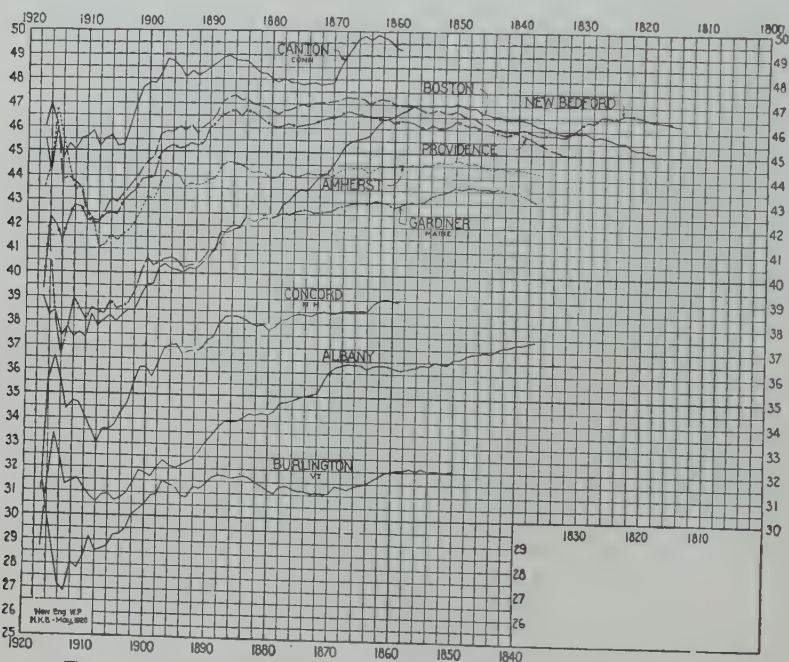


FIG. 15.—Precipitation in New England—variation in yearly mean.

Range in Distribution of Mean Monthly Precipitation.—The total amount of the annual rainfall is only one of the elements that influence the run-off. The time of occurrence, or the periodical distribution of the rainfall, is even of greater importance. The general character of the periodical distribution of the rainfall tends to be similar each year in each locality, and the maximum and minimum monthly rainfalls occur in this occurrence from year to year, a consideration of the records over a number of seasons, or the mean of several years, shows that the same general characteristics ordinarily prevail for any given month in a given section. The character of this monthly distribution varies widely in

¹ See *Jour. Boston Soc. Civil Eng.*, January, 1921, p. 5.

different parts of the country. Thus the New England and north Atlantic states present a general similarity in the distribution of monthly rainfall. Similarly, the Ohio Valley constitutes another area; the Great Plains west of the Mississippi still another area, and the Pacific slope an additional one in all of which there are typical and different distributions of monthly rainfall during the year.

In New England the maximum precipitation occurs ordinarily during the summer months, July and August, but during no one month is there any great variation from the mean. The Ohio region is similar to New England but shows a tendency toward maximum precipitation in May and June and a slightly greater variation from the mean. Along

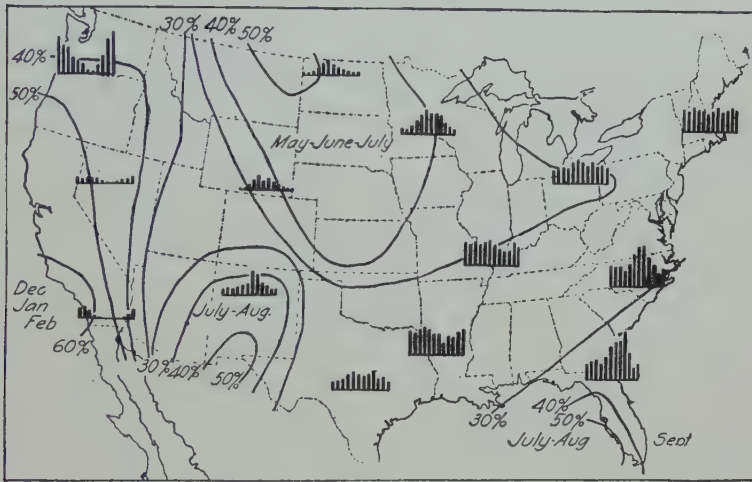


FIG. 16.—Variation in monthly rainfall of United States.

and near the south Atlantic coast the summer months show a maximum precipitation with a decided variation from the mean. The Great Plains and north central states show a maximum in June and a minimum in January, with a fairly regular variation between those limits during the remainder of the year, some 18 per cent of the yearly precipitation occurring during June and only about 2 per cent during January. On the Pacific coast the minimum occurs in the months of July and August, and there is a fairly uniform increase in both directions, reaching a maximum in December or January. Here the minimum month shows less than 1 per cent of the yearly precipitation (in some cases none whatever), while during the maximum month over 20 per cent occurs.

The variation in monthly precipitation in different sections of the United States is shown in Fig. 16¹ as well as the percentage of annual precipitation commonly falling in the wet season of the year.

¹ MEAD, "Hydrology," Fig. 125.

As will be further noted, the distribution of precipitation during the different months of the year has a great effect upon stream flow available for water power, owing to the much greater evaporation, transpiration losses, etc., which occur during the growing or summer season, when much of the precipitation does not reach the streams. Obviously, the typical precipitation of distribution of the Great Plains, coming mostly during the summer months, while favorable for agriculture, tends toward a low stream flow and conditions unfavorable for water power use.

Extremes of Monthly Precipitation.—Single months will show great variations in precipitation. Thus, at *Boston* (with a monthly average of 3.65 in.) in March, 1915, no measurable amount occurred; in October, 1924, only 0.06 in. was recorded; and minima of 0.2 to 0.5 in. have been reached by all months but July. A maximum of 12.38 in. was recorded in July, 1863, and many other months have shown maxima over 10 in. During the 100 years of record no precipitation equal to the monthly maximum has occurred since 1877, while minima for 4 months of the year have occurred since 1911.

Washington, D. C., exhibits about the same range of monthly extremes as Boston, varying from about 0.2 to 13 in. and averaging 3.40 in.

Leavenworth, Kan., with a monthly average of 2.90 in., for 73 years, shows 8 months of the year in which a precipitation of 0.10 in. or less has occurred (in 73 years of record), while a maximum of nearly 16 in. has occurred (in June, 1845), for which month the average is 5.15 in. About 70 per cent of the average yearly precipitation occurs in the 6 months April to September, inclusive, which makes, however, owing to the lesser yearly total, only a little more precipitation than occurs at Boston during these same 6 months of the year.

San Francisco (averaging 1.90 in. per month for a record of 50 years), as would be expected, shows many months with no precipitation—confined mostly to June, July, and August—all of which average only 0.02 in. May, September, and October are also months of low precipitation with occasionally no measurable amount. The month of maximum precipitation is January, averaging 4.82 in. and reaching a maximum of 24.36 in. in 1862. Maxima of 11.78, 15.16, and 12.52, in. have also been recorded in November, December, and February, respectively. Over 90 per cent of the average yearly precipitation occurs in the 6 months, November to April, inclusive.

These few illustrations show some of the great fundamental differences in precipitation in different sections of the country which directly affect stream flow and, therefore, the amount and seasonable distribution of water available for power purposes.

Distribution of Precipitation during the Month.—Thus far only yearly and monthly total precipitations have been considered. It is evident that the distribution of any given amount of precipitation during the

month will greatly affect the resulting stream flow, particularly during the growing or summer season. Numerous showers, all small in amount, may contribute only enough water to take care of evaporation and vegetation demands, etc., whereas the same monthly total concentrated in one or two storms, while not so favorable for crops, will provide more water for the streams. Any careful study of precipitation to use as a basis for estimating or checking stream-flow estimates may, therefore, require a study of the daily records to be of value.

During the winter months, in the colder latitudes where snow storage occurs, the daily distribution of precipitation is not of especial importance as affecting stream flow, for, as will be further noted, the condition of the ground surface, variation in temperature, etc., are of more importance. In fact, during any given winter month there may be no definite relation between the amount of snowfall and stream flow.

Precipitation and Altitude.—The relation of precipitation to altitude is a subject of frequent discussion, and it has been found that there may be an increase or a decrease in the amount of precipitation as altitude varies, depending upon the part of the country considered. For the portions adjacent to the oceans, as would be expected, there is a decided tendency toward increase in precipitation with increase in altitude. Thus, at the summit of Mt. Washington records kept by the U. S. Weather Bureau extending over 15 years give an average yearly precipitation of about 82 in., or about double that at the ordinary level of the river valleys in the general vicinity of Mt. Washington. Similarly, in the southern Appalachian Mountains the annual precipitation reaches 70 in. or more over a comparatively small area high in elevation, while near the coast the precipitation is only about 40 to 45 in. Near the Pacific coast as well, and again at the Rocky Mountain range, a similar relation is found to hold.

On the other hand, in the Mississippi Valley and the central states the general tendency is toward diminution in the amount of precipitation going northward from the Gulf of Mexico. Whereas at the Gulf of Mexico there is a mean annual precipitation of 60 in. or more, in northern Minnesota this is reduced to from 20 to 30 in. Going northward there is, of course, a gradual increase in altitude of the land surface, but the air currents coming in from the Atlantic and Gulf ordinarily become diminished in their supply of moisture before they reach these higher altitudes in the upper Mississippi Valley.

After crossing the summit of a mountain range, precipitation is likely to decrease markedly, as shown clearly in Fig. 17, in which is shown a profile normal to the coast line through the Sacramento Valley, California, accompanied by a curve of mean annual precipitation.¹ The western

¹ LEE, C. H., "Yield of Underground Reservoirs," *Trans. A.S.C.E.*, vol. 78, Plate II, 1915.

slope, increasing in altitude to about 6000 ft., shows a steadily increasing precipitation to a maximum at about the lower cloud limit. Above this point the latent heat of condensation is sufficient to raise the temperature above the dew point and cause a decrease in precipitation. Beyond the crest of the Sierras, a situation occurs often called the "rainfall shadow," where even though the elevation remains at about 5000 ft., precipitation falls off to a small amount, as the descending mass of air contracts in volume and keeps its temperature above the dew point.

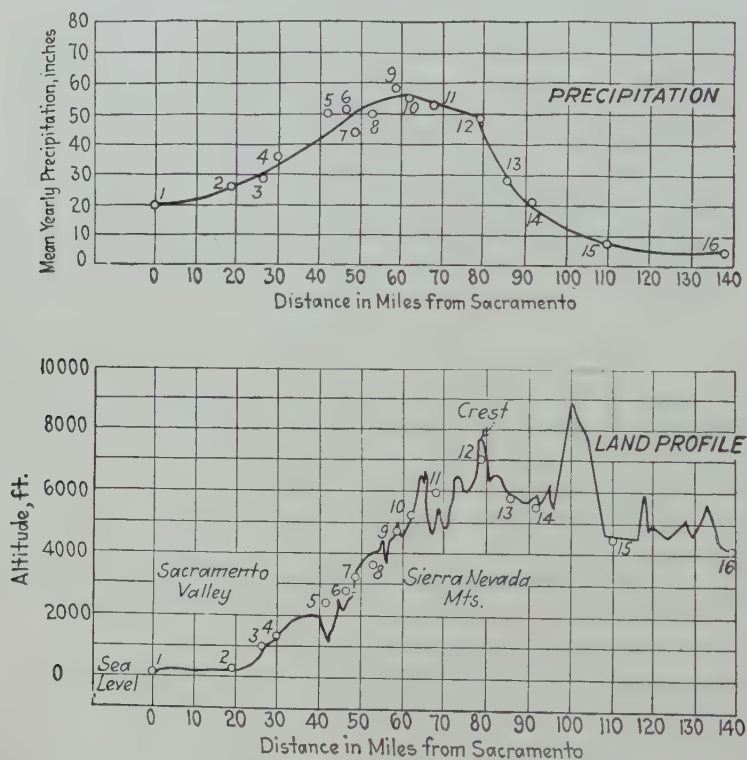


FIG. 17.—Precipitation and altitude in California.

There is, therefore, a general and fundamental tendency for precipitation to increase as altitude increases on the coastal side of mountain ranges, provided the locality is not too far from the general sources of water vapor so that its previous depletion near the coast or in elevated regions may have already occurred.

Our information regarding precipitation in the higher altitudes is meager and unsatisfactory, and apparent discrepancies between data of precipitation and run-off may often be traced to an incorrect estimate of precipitation based upon results at stations low in altitude compared with the average altitude of the drainage basin in question. It is, there-

fore, often important to consider the relative elevation of precipitation stations and of the drainage area being studied, if correct conclusions are to be drawn relative to precipitation.

The relation between precipitation and altitude may be studied to a limited extent by plotting curves of altitude precipitation. Thus, for stations on the coastal side of the White and the Green mountains in New England, an approximate relation of $P = 35 + 7.5A$ has been deduced,¹ where P = mean annual precipitation in inches, and A = altitude in thousands of feet.

S. A. Hill² found that in the northwest Himalayas, where rainfall is remarkable in amount and variation, the following formula applied: $\text{Rainfall} = 1 + 1.92h - 0.40h^2 + 0.02h^3$, where h is the relative height in units of 1000 ft. above an assumed plane 1000 ft. above sea level. This gives a maximum rainfall at an elevation 4160 ft. above sea level. He also found a rapid diminution in rainfall on the leeward side of the mountains, about half that for stations of equal elevation on the windward side.

It must be kept clearly in mind that such relations between altitude and precipitation are always local in character and must, therefore, be developed and used locally and with caution to avoid gross errors in conclusions. They are often of help, however, particularly in studying variation in precipitation and stream flow on a coastal drainage area where a considerable range in altitude exists.

Mean Precipitation on a Drainage Area.—It frequently is desirable to determine the mean precipitation for a given drainage area over some period of time, such as for a particular storm, month, year, or long period, etc. This may be done by plotting "isohyets" or lines of equal precipitation properly interpolated from available records and obtaining a weighted average by planimeter.

A simple graphical method often used, known as Thiessen's method,³ is based upon the assumption that the record at any one station should be used for the portion of the drainage area *nearest* that station. As will be seen from Fig. 18 where A, B, C, D are precipitation stations to be used in determining the mean amount upon an adjacent drainage area, by erecting perpendiculars halfway between adjacent stations the portions of the area nearest any particular station may readily be defined and in the figure are marked a, b, c, d . The percentage of the total area corresponding to a, b, c, d , determined by planimeter, is then applied to the

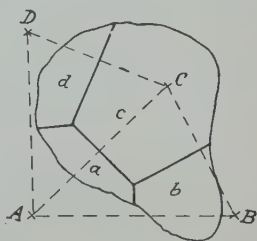


FIG. 18.

¹ See *Jour. Boston Soc. Civil Eng.*, January, 1921, p. 4.

² U. S. Geol. Survey, *Water Supply Paper* 81, p. 354.

³ *Jour. New Eng. Water Works Assoc.*, March, 1924, p. 26.

respective station records *A, B, C, D*, the sum of these amounts being the proper weighted precipitation for the drainage area.

In another method¹ called the center-of-gravity method, weights are assigned to the different records inversely proportional to the distances of the several stations from the center of gravity of the drainage area. This method appears to be inferior, however, to that previously given.

In determining the mean precipitation for a given area, the records at individual stations should either include the same years or time of record or, when determining a long-time mean, preferably be modified by comparison with some long-time or index record near by, assuming the relation that is found between the short- and long-time record at the index station to hold for the other stations.

Supplying Missing Data for Precipitation.—Often, particularly in the older records, one or more months' records may be missing. These may be approximated from adjacent records, preferably by using three stations forming a triangle which includes the station for which the estimate is to be made, upon the assumption that the ratio of the record at the latter for the missing month to the mean for that month will be the same as a similar ratio for each one of the surrounding stations. The mean of the three values thus estimated from surrounding stations will usually be reasonably close to the probable precipitation at the enclosed station for the missing month. Where three adjacent stations are not available, two or even one station may be used in a similar manner. The results of such computations must be used with caution but are frequently of value in order to obtain complete records for comparative purposes over a period of years.

Maximum Rainfalls.—In the study of flood possibilities and the necessary provision to insure the safety of a water power development against damage due to high water, the study of maximum rainfalls is of fundamental importance. The actual flood or maximum run-off from the drainage area in question would be the best index of what to prepare for in the future, but such information is often lacking and it becomes necessary to study and use data of flood flows on adjacent or similar streams or to estimate probable floods from records of unusual rainfall.

Three factors are important in making use of data of maximum rainfall for floods estimates, *viz.*: (1) the amount of rainfall for given periods of time, as 1, 2, 3 days, etc., (2) the area over which the rainfall occurs, and (3) the frequency with which maximum rainfalls are likely to occur.

Unless a drainage area is quite small, the maximum rainfalls for time periods of 1 or more days will usually serve the purpose of estimates, at least in the usual water power development. For small drainage areas

¹ *Ibid.*, p. 28.

(and especially in problems aside from water power development, such as the design of storm-water sewers), it will be necessary to consider shorter periods of time than a day.

At most of the rainfall stations the ordinary rain gage is used and read once in 24 hr., thus giving no information as to hourly rates and often in a large storm not even giving the maximum 24-hr. amount, which may be for portions of two successive days. There are, however, several hundred recording gages, some of which have been established for a considerable time, and much information is even now available.

A very elaborate study of maximum rainfalls has been recently made by the engineers of the Miami Conservancy District, Dayton, Ohio.¹ All station records of five consecutive years or more in the United States east of the 103° of longitude, the easterly boundary line of New Mexico (about 3000 stations), were included through the period 1870-1914, with the following limitations:

Where the normal annual precipitation exceeded 20 in., they considered only storms where the total daily rainfall equaled 10 per cent or more of the normal annual, or where the total storm rainfall equaled 15 per cent of the normal annual.

Where the normal annual precipitation was less than 20 in., they considered storms where there was a total rainfall of 4 in. or more.

For convenience the United States was divided into 2-deg. quadrangles bounded by the odd-degree meridians and parallels, and as one feature of the investigation, maps were prepared showing maximum 1- to 6-day rainfalls in each quadrangle (*ibid.*, pp. 103-105). A few of these results are given in Table 23.

TABLE 23.—MAXIMUM RAINFALLS IN THE EASTERN UNITED STATES

Quadrangle	Number of stations	Approximate location	Maximum rainfall, inches					
			Days					
			1	2	3	4	5	6
3-D	75	{ Massachusetts Rhode Island Connecticut }	11.8	14.6	14.7	14.7	14.7	14.7
4-D	47	New York	9.7	11.8	12.1	12.7	12.7	12.7
9-E	50	Ohio	6.5	9.5	10.4	11.1	11.4	11.7
14-D	38	Iowa	14.6	14.7	15.5	15.5	15.5	15.5
8-I	13	Georgia	10.4	13.5	14.1	14.3	16.9	17.8
16-I	22	Texas	11.0	13.2	15.0	15.7	15.7	15.7
8-J	24	Florida	18.0	19.1	19.1	19.1	19.1	19.1

¹ "Storm Rainfall of the Eastern United States," Miami Conservancy Dist., Dayton, Ohio, *Tech. Report*, Part V.

Area Covered by Maximum Rainfalls.—It must be kept clearly in mind that the foregoing table and similar data referred to in map form are of maximum rainfalls over relatively small areas and it is, therefore, necessary to consider the areas covered by the greater storms and the variation in amount of rainfall within those areas.

One of the earliest investigations made of storm area and intensity was that by James B. Francis of the Oct. 3-4, 1869, storm in New England.¹ This storm is the largest on record in New England within the last 50 years and covered parts of two calendar days. The greatest measured rainfall was 12.35 in. at Canton, Conn., and several stations indicated more than 8 in. in 36 hr. No data were available to give the 24-hr. rainfall. The areas covered for different rainfalls were as follows:

Rainfall, Inches	Area Covered, Square Miles
6 or more.....	24,000
7.....	9,600
8.....	1,800
9.....	1,050
10.....	520
11.....	180

In the *Report of the Miami Conservancy District on rainfall* previously referred to is also compiled much valuable information upon the area covered by different storms. From the tables on pages 204-207 of that report the data in Table 24 are compiled.

Three of the storms listed in Table 24 are especially noteworthy—those of Ohio in March, 1913, and of Alabama and of North Carolina in July, 1916.

The March, 1913, Ohio storm is of especial interest as one of the greatest storms on record in the eastern United States which caused unparalleled flood damage aggregating a monetary loss of some \$200,000,000. While no exceptional 24-hr. rates of rainfall were observed, a record for continued rainfall was established at a number of places. The unusual duration of heavy precipitation was brought about by the merging of storms accompanying two distinct low-pressure areas, which occurred almost simultaneously in a low-pressure trough extending from southwest to northeast. One of these areas progressed from Iowa in a northeasterly course across the upper Lake region, and the other followed it 2 days later, moving along a parallel route from central Arkansas toward Lake Erie. The bulk of the precipitation in the lower Ohio basin fell during the 72-hr. period, March. 24-26. The extent of this storm was very great, the 2-in. isohyetal extending from Oklahoma to Maine and including an area of about 492,000 sq. miles. The average depth of this

¹ *Trans. A.S.C.E.*, vol. 7, pp. 224-235, 1878.

area was 4.2 in. for the 5-day period. The 6-in. isohyetal covered about 53,000 sq. miles and the 8-in. about 15,000 sq. miles (see Fig. 19).

The July, 1916, storms in the southern states were extraordinary in that two tropical hurricanes occurred during one month, first in the

TABLE 24.—EXTENT OF RAINFALL IN GREAT STORMS—UNITED STATES

Storm	Center	Period, hours	Greatest rainfall in inches over area in square miles					
			1	500	1000	2000	4000	6000
Oct. 3-4, 1869.....	Connecticut	48	12.4	10.4	9.7	8.9	8.1	7.8
July 12-14, 1897....	Connecticut	24	8.0	7.3	6.8	6.2	5.6	5.2
		48	10.3	9.5	9.1	8.5	7.5	6.9
		72	10.3	9.6	9.3	8.6	7.7	7.0
Oct. 8-9, 1903.....	New Jersey	24	11.5	9.3	8.4	7.6	6.8	6.3
		48	15.0	11.9	10.9	9.9	9.0	8.4
Mar. 23-27, 1913....	Ohio	24	7.0	6.7	6.5	6.2	5.8	5.7
		48	9.5	8.8	8.4	8.2	8.0	7.8
		72	10.2	9.7	9.5	9.2	9.0	8.7
		96	11.1	10.4	10.0	9.6	9.4	9.3
		120	11.2	10.9	10.8	10.5	10.1	9.9
Aug. 25-28, 1903....	Iowa	24	11.2	10.6	10.0	9.1	7.8	6.8
		48	14.7	11.8	10.8	9.7	8.3	7.7
		72	15.5	12.2	11.4	10.5	9.4	8.7
		96	15.5	12.4	11.7	10.7	9.6	8.8
June 9-10, 1905.....	Iowa	24	12.1	10.9	10.0	8.9	7.8	6.9
July 14-16, 1916.....	North Carolina	24	19.3	12.2	11.3	9.9	8.4	7.5
		48	23.2	18.4	17.1	15.5	13.7	12.5
		72	23.7	18.4	17.3	15.6	13.8	12.7
June 29-July 3, 1909.	Florida	24	12.0	11.5	10.3	8.8	7.6	6.8
		48	15.5	14.2	13.0	11.8	10.8	9.9
		72	15.8	14.6	13.4	12.4	11.4	10.5
		96	17.7	15.4	14.9	14.1	12.9	12.0
		120	17.8	16.4	16.0	15.5	14.8	14.2
July 6-10, 1916.....	Alabama	24	11.1	10.7	10.2	9.5	8.8	8.2
		48	17.3	15.8	14.8	13.7	12.3	11.3
		72	19.7	18.7	17.8	16.5	14.7	13.1
		96	20.3	20.0	19.8	19.3	18.4	17.4
		120	22.3	20.4	19.9	19.4	18.6	17.7
Aug. 17-20, 1915....	Texas	24	9.1	8.7	8.0	7.9	7.7	7.5
		48	18.6	16.2	15.0	13.6	12.3	11.7
		72	19.7	19.6	19.5	19.2	18.6	17.9

period from the sixth to the tenth, centering in Alabama, and again during the fourteenth to the sixteenth, centering in the Carolinas.

The first of these entered the United States just east of the Mississippi River on July 5 (see Fig. 20 and Table 24) and resulted in very high rainfalls, especially for the 3- and 4-day periods. Much damage resulted,

especially in Alabama, where property loss was estimated at about \$11,000,000.

The storm of July 14-16, 1916, was produced by a West Indian hurricane or tropical cyclone which approached the south Atlantic seaboard from the southeast, entering South Carolina near Charleston on the morning of July 14. As is commonly the case with hurricanes after they come in contact with land surfaces, this storm rapidly diminished in

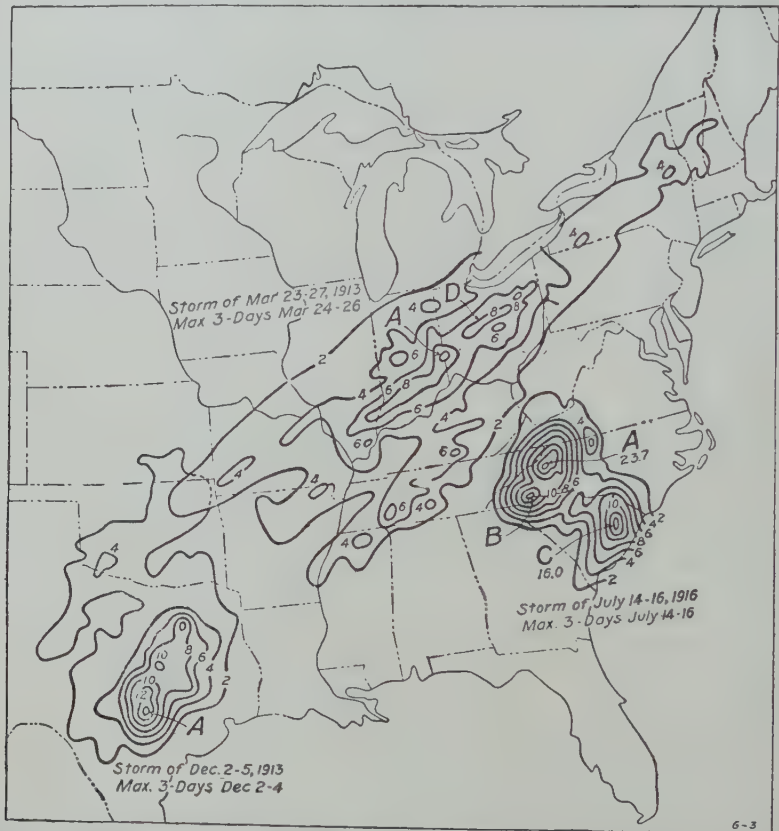


FIG. 19.—Rainfall map—storms of March and December, 1913, and July 14 to 16, 1916.

energy and finally dissipated on July 16 on the eastern slope of the mountains in North Carolina. This illustrates the important influence which high mountain ranges exert in modifying or arresting the progress of cyclonic movements. Statistics published by the Weather Bureau tend to show that the tracks of cyclones do not as a rule cross the Appalachian Mountains.

Two distinct centers of precipitation formed during this storm, one over South Carolina with a peak of over 16 in. in 2 days, and the other over North Carolina with a peak of over 23 in. in 2 days. The maximum

rainfall in the latter was recorded at two stations, about 1 mile apart, located on opposite sides of a mountain gap, nearly midway between Grandfather Mountain, elevation 5964 ft., and Mt. Mitchell, elevation 6711 ft. These stations are Altapass, elevation 2625 ft., on the east slope, where a standard 8-in. rain gage indicated a total fall of 23.77 in. for the 3 days July 15–17, and Altapass Inn on the west slope, where the total fall was 1.52 in. less. At Altapass 23.22 in. of rain fell between

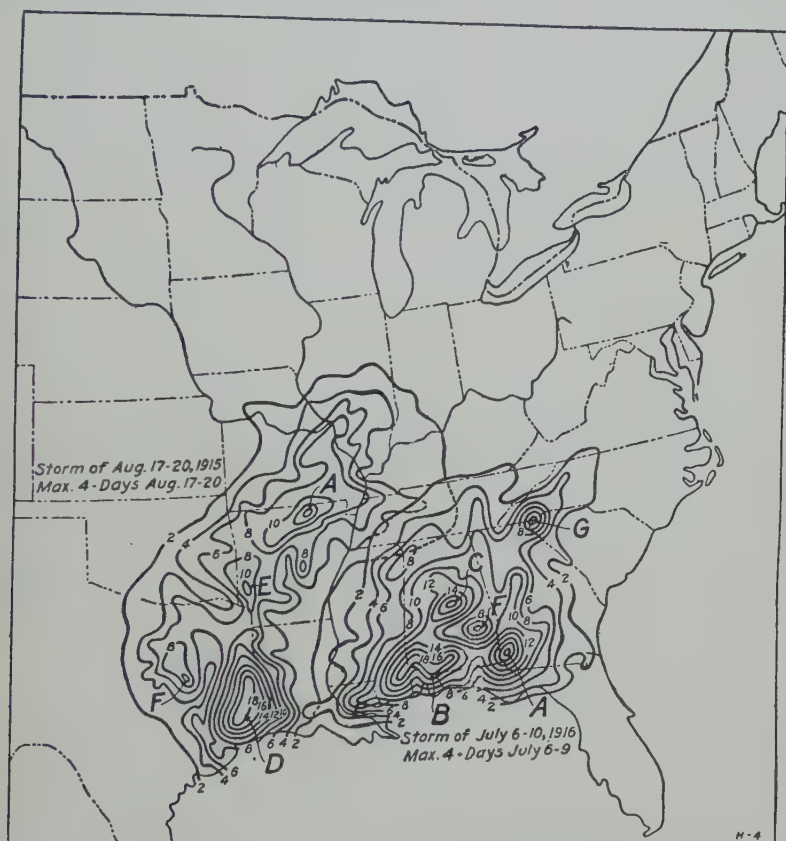


FIG. 20.—Rainfall map—storms of August, 1915, and July 6 to 10, 1916.

2 P.M. of July 15 and 2 P.M. of July 16—one of the heaviest records in this country.

To the westward of the mountains the rainfall decreased rapidly as indicated by the isohyets in Fig. 19, the total fall at points 40 miles west of Altapass being less than 1 in.

As this storm progressed in a westerly direction, the lower portions of the longer Atlantic coast streams were in flood in many cases before their headwaters, resulting in floods less severe than would normally be

expected from such rainfall. On the other hand, these floods were due in some measure to the saturated condition of the soil, brought about by the previous heavy rains which occurred in this region earlier in the month.

The time-area-depth relations for these two remarkable storms are given in Table 24 and their isohyets in Figs. 19 and 20.

The New England storm of Nov. 3-4, 1927, was noteworthy as to both rainfall and resulting flood damage.¹ The storm was of the Atlantic coastal type, of tropical origin, with its principal center in Vermont and western Massachusetts but a secondary center in Rhode Island.

The greatest observed rainfall was 9.65 in. at Somerset, Vt., in about 36 hr., while Westerly, R. I., recorded 9.37 in. In the higher elevations of the Green Mountains the storm rainfall was estimated at 10 to 12 in. The rainfall-area relation based upon station records was as follows:

Rainfall, Inches	Area, Square Miles
5	22,030
6	8,470
7	4,040
8	1,795
9	497

The greatest damage—about \$30,000,000—occurred in Vermont, where steep river and land slopes, supplemented by wet conditions due to previous rains, caused very rapid run-off and the loss of 84 lives as well as great damage to railroads, highways, bridges and buildings, and to farm lands and stock.

A study of the data given in Table 24 indicates the results as shown in Table 25, as the approximate maximum rainfalls which may be expected to occur sooner or later (1) in the northern and central states (east of the 105th meridian) and (2) in the southern coastal states.

These values are only approximate but will serve reasonably well to define in a general way limiting values of rainfall in these districts. West of the 105th meridian no comprehensive study has as yet been made to serve as a basis for similar data applicable to that part of the country.

Frequency of High Rainfalls.—The third factor of importance in the study of maximum rainfalls is that of frequency, which may be defined as the number of times within a selected period of years that any particular amount of rainfall has occurred. Dividing the period by the number of such occurrences would give the average length of time in years during which the particular rainfall has occurred once, which is also (less accurately) sometimes called frequency, keeping in mind, however, that this does not mean any regular or stated interval of occurrence.

¹ "The New England Flood of 1927," U. S. Geol. Survey, *Water Supply Paper* 636-C, 1929; also "Report of Committee on Floods," *Jour. Boston Soc. Civil Eng.*, September, 1930.

TABLE 25.—APPROXIMATE MAXIMUM RAINFALLS

Time, hours	Greatest rainfall in inches over area in square miles		
	1	500 to 1000	5000
	Northern and central states		
24	12	10	7
48	15	12	8
72	15	12	9
96 and over	15	12	9
	Southern (coastal) states		
24	19	12	9
48	23	15	12
72	23	18	14
96 and over	23	20	17

A thorough study in rainfall frequency was made in Part V of the *Report* of the Miami Conservancy District previously quoted. To illustrate the process used, considering quadrangle 9-E (near the Miami River Valley, Ohio), there were 28 rainfall stations with an aggregate period of record of 713 years (excluding stations with less than 10 record years). The eight greatest 1-day rainfalls in this quadrangle were as follows in order of magnitude:

		Inches
1	Newport, Ky..... May 24-25, 1858.....	6.35
2	Urbana, Ohio..... Sept. 18, 1866.....	6.20
3	Cincinnati, Ohio..... June 17-18, 1868.....	6.00
4	Urbana, Ohio..... June 15, 1868.....	5.95
5	Bellefontaine, Ohio..... Mar. 25, 1913.....	5.61
6	College Hill, Ohio..... June 18, 1875.....	5.50
7	North Lewisburg, Ohio..... Sept. 27-28, 1884.....	5.40
8	Newport, Ky..... Aug. 14, 1850.....	5.40

From the foregoing table it appears that an intensity of 5.40 in. in 1 day has been equaled or exceeded and is, therefore, likely to be equaled or exceeded in future years, on an average, once in $713\frac{1}{4}$ = about 100 years at any point in the quadrangle.

The figure 5.40 was called the "pluvial index" for the quadrangle corresponding to a 100-year period and a 24-hr. rainfall, and similar indexes were computed for 2- to 6-day periods and times of 15, 25, 50, and 100 years and plotted on "isopluvial charts" for the eastern United States, on which were also drawn for convenience "isopluvial lines."

As would be expected, these charts show a high pluvial index along the Gulf coast, diminishing rapidly toward the interior, except in the Mississippi Valley, where the lines extend markedly northward. The dominating feature is the general decrease in pluvial index with increase of latitude. There is also a decided lowering as altitude increases going westerly in the Great Plains district.

Another interesting fact, which may be seen by comparing these isopluvial charts with maps of mean annual precipitation, is that a high annual precipitation does not necessarily mean high pluvial indexes, as shown by the southern Appalachian Mountain area, where the annual rainfall reaches 80 in., but no corresponding increase appears in pluvial indexes.

It must be kept in mind that these pluvial index figures will be modified as further rainfall records are available, a small amount probably in such a quadrangle as 3-D (Massachusetts, Rhode Island, and Connecticut) where 1508 years of record were used, and to a much greater extent in localities like 17-G (Texas and Oklahoma) where only 110 years were used. Furthermore, the pluvial index is of course more accurate for the shorter times, as 15 years, than for the 100-year periods.

For any particular quadrangle, frequency curves may be drawn, plotting as abscissas "average number of years between periods of excessive precipitation" and as ordinates, "average depth in inches," with a curve for each time period, as 1 day, 2 days, etc. Typical frequency curves are thus given in Figs. 44 to 47 inclusive of the report, from which are abstracted the values given in Table 26.

TABLE 26.—FREQUENCY OF EXCESSIVE RAINFALLS

Quadrangle	Average number of years between periods of excessive rainfall	Average depth in inches for periods of days			
		1	2	4	6
3-D (Massachusetts, Rhode Island, and Connecticut)...	20	4.5	5.0	5.8	6.2
	50	5.5	6.6	7.3	7.6
	100	6.5	7.6	8.1	8.5
9-E (Ohio).....	20	4.2	5.4	7.0	7.3
	50	5.0	6.5	8.2	8.9
	100	5.5	6.9	8.8	9.3
12-J (Louisiana and Mississippi).....	20	7.8	9.0	10.3	11.5
	50	8.9	9.8	11.5	12.3
	100	9.2	10.2	12.2	14.4
15-E (Kansas and Nebraska).	20	5.4	6.0	7.1	8.1
	50	6.5	7.1	8.5	9.6
	100	7.5	8.1	9.3	10.4

In the above table the depth corresponding to any frequency period is that which will probably be equaled or exceeded once during that period. An inspection of the above table (or better still of the curves from which the table is prepared), as compared with the data of maximum rainfall as given on page 65, indicates that these maxima are not in general likely to be greatly exceeded in the future. For illustration, the maximum of 11.8 in. for a 1-day rainfall in quadrangle 3-D (see page 70) is seen to be of very rare occurrence. An amount of 6.5 in. represents what may be expected to be equaled or exceeded once in 100 years, while the 11.8 in. is found only once in records aggregating some 1500 years.

It must be kept clearly in mind, however, that rainfall frequency as discussed above is simply an estimate of the average time interval of occurrence, based upon past performance. There is nothing to indicate how soon a rainfall equal to or greater than any of these tabular values may actually occur. As will be noted further, under estimates of flood flows, such studies of frequency are useful in aiding the judgment to decide what factor of safety is proper in the design of works to resist and handle flood waters.

Where to Get Precipitation Data.—Data of precipitation are to be found primarily in the reports of the U. S. Weather Bureau, which can be consulted at libraries and at the local weather bureau offices. The published records are in the form of monthly reports, giving the detailed and total precipitation for each station. At the end of each year a summary is published, which contains the information on precipitation usually required in water power studies.

In 1908–1909 the Weather Bureau published summaries of practically all of its precipitation records, giving the precipitation by months and years for the entire period available. The country was divided into 105 different sections, and a pamphlet issued for each one of these sections which contains data upon precipitation, relative humidity, winds, and other meteorological phenomena. This very useful summary, known as *Bulletin W*, contains all the information regarding precipitation available up to either 1908 or 1909 for the United States. The various sections of this summary have been revised to include data through the year 1921.

For New England very complete summaries of monthly rainfall have been published through 1928.¹

Some of the special reports upon water resources by the U. S. Geological Survey in the *Water Supply Papers* also contain data of precipitation. Various municipal and state organizations also frequently publish precipitation data of local interest.

¹ GOODNOUGH, "Rainfall in New England," *Jour. New Eng. Water Works Assoc.*, June, 1930, pp. 157–351.

The student should familiarize himself particularly with the published data of the Weather Bureau as the best general source of information on this subject.

DISPOSAL OF PRECIPITATION

Of the precipitation which falls upon the drainage basin of a stream, a portion is evaporated directly by the sun; another large portion is taken up, in the growing season, by vegetation and growing crops and mostly transpired in vapor; still another portion, large in winter but very small in summer, finds its way over the surface directly into the stream, forming surface flow. Finally, another part, called percolation, sinks into the ground to replenish the great reservoir of ground water from which plants are fed and stream flow maintained during the periods of slight rainfall, for the rainfall is frequently, for months together in the summer, much less than the combined demands of evaporation, plant growth, and stream flow. These demands are inexorable, and it is the ground storage which is called upon to supply them when rain fails to do so.

The combined requirements of evaporation, transpiration, etc., may properly be called water losses, although often referred to as "evaporation," "retention," etc.

Surface flow is, therefore, that portion of the precipitation remaining after water losses and percolation requirements are supplied. Stream flow or run-off is made up of surface flow plus water of percolation, which is constantly entering the stream from the adjacent ground water along its course, as well as from occasional springs or visible outlets of ground water. In the broad sense, therefore, stream flow consists of precipitation minus water losses, subject to the effects of the lag of percolation and seasonal variation in level of ground-water storage.

Attempts have frequently been made to express in equational form a relation between stream flow and precipitation. While such relations may sometimes be worked out for average yearly or seasonal periods on a particular drainage area, they must be based on actual measurements of both factors, which would not necessarily be even approximately correct when applied to some other drainage area, the obvious purpose of such formulas. It may well be said that every stream is a law unto itself as regards flow, and this law can best be determined by actual measurements of its flow over a considerable period of time. Such measurements in adequate extent are often not available, however, at water power sites and it may be necessary to use data of flow of other streams, or even of precipitation, in estimating or checking up the probable flow at the site considered.

An intelligent understanding of the factors which affect the disposal of precipitation is fundamental in such studies, and these will now be considered.

Factors Affecting Disposal of Precipitation.—The manner of disposal of precipitation may be outlined as shown on page 74. The factors which vary the proportion between stream flow and water losses are also tabulated. As will be noted, these are classed under three general headings, *viz.*: (1) meteorological conditions, (2) drainage-area characteristics, and (3) storage.

1. *Meteorological Conditions.* Precipitation.—This has already been discussed in detail but not with reference to its disposal as stream flow and water losses. It will be evident, however, that these elements will both be affected by the following factors:

The amount of precipitation, as the source of water for both stream flow and water losses.

The intensity of precipitation, a high intensity tending to saturate the ground and cause a greater proportion of surface flow, due to water moving so rapidly that water losses as well as percolation may not have time to occur.

The area covered by the precipitation in a given storm, which in the case of a large drainage area may affect it only in part. Conversely, a small drainage area may be wholly included in the area of maximum rainfall of a severe storm, resulting in excessive surface flow and floods.

The seasonal distribution of precipitation, which if occurring mostly during the growing season will result in excessive water losses and, hence, a small stream flow. Conversely, the winter season favors a high proportion of stream flow.

The proportion of snow and rain, a large snowfall tending in general toward a greater proportion of ultimate stream flow due to snow storage, concentrated melting, and quick run-off.

Temperature.—Aside from its basic importance in influencing the amount of precipitation, temperature also affects its disposal as follows:

Evaporation and transpiration losses, as will be further discussed, increase with temperature and are at a maximum during the summer season and a minimum in winter.

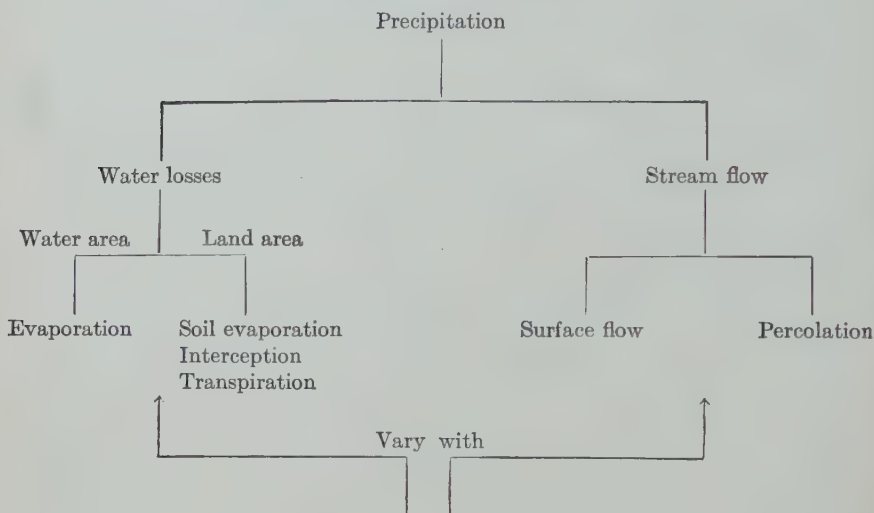
Snow storage, the result of more or less continuous temperatures below freezing, profoundly affects the regimen of stream flow. In the northern states this results in a deferred run-off, usually occurring more or less concentrated near the end of the winter season. Where drainage areas extend to mountainous regions, this deferred run-off may occur late in the spring or even early summer, and for altitudes above 7000 to 8000 ft. in the West—the regions of perpetual snow and glaciers—this melting may contribute largely to stream flow during the summer season.

Frozen ground, the result of low temperatures without snow cover, may temporarily render a drainage area nearly impervious, so that sudden rains may cause excessive run-off.

Relative Humidity.—A measure of the extent to which the air contains water vapor, and consequently an indication of its capacity to

absorb more moisture evaporated from land and water surface, is of importance in affecting evaporation, as will be further discussed.

DISPOSAL OF PRECIPITATION AND FACTORS AFFECTING ITS DISPOSAL



1. Meteorological conditions:

Precipitation: Amount

Intensity

Area covered

Seasonal distribution

Proportion of snow and rain

Temperature: Affects evaporation and transpiration losses

Snow storage

Frozen ground

Relative humidity: Affects evaporation

Wind, amount of: Affects evaporation

2. Drainage-area characteristics:

General: Size of drainage area

Shape of drainage area

General location

Topography: Surface slopes

Water area: Lakes or ponds, swamps

Geology: Character of surface

Character of subsurface

Condition: Cultivation

Vegetation

Drainage

3. Storage: Natural lakes and ponds

Artificial reservoirs

Ground storage

Wind.—The velocity of wind is also of importance in affecting the amount of evaporation as well as interception losses.

2. *Drainage-area Characteristics.* General.—The size of drainage area is of particular importance in respect to extremes of run-off. A small area tends to have a higher unit (or per square mile) flood discharge than a large area, as has already been explained. The smaller area may also give a lower unit run-off in the dry season, due to local conditions which may be compensated on a large area.

The shape of drainage area also affects flood tendency, a fan-shaped arrangement of tributaries often resulting in high flows reaching the main stream at about the same time, while a long and relatively narrow area would not have this tendency.

The general location may also affect flood tendency, particularly where a large area lies in a general direction paralleling the usual path of storms.

Topography.—Surface slopes are important in affecting the disposal of precipitation, steep slopes tending toward a rapid and large surface flow. Such slopes are also commonly found where the surface material is relatively hard and impervious so that little percolation occurs. Flat areas and gentle slopes, on the other hand, tend to diminish surface flow and increase water losses as well as percolation.

The amount of water area, in lakes, ponds, and swamps, affects losses, as in general evaporation losses from water area exceed the land water losses. A large percentage of water area, therefore, tends to a diminished run-off.

Geology.—The character of surface or soil conditions, whether pervious or not, also of subsurface conditions, the depth of pervious strata, and the character of underlying strata will obviously affect the amount of percolation as well as water losses. A rather flat drainage basin with pervious soil will tend toward moderate floods and a well-sustained flow during the dry season. A basin with steep slopes and impervious soil will be "flashy" in its characteristics, with greater flood tendencies as well as a lower minimum flow.

Condition of Area.—The condition of the drainage area as to cultivation and vegetation will affect the amount of water losses as well as percolation. As will be further discussed, soil evaporation and percolation are increased by cultivation, while transpiration and interception losses are occasioned by the presence of vegetation.

The introduction of artificial drainage will tend to modify run-off, changing the amount of surface storage in swamps, meadows, etc., and, where canals and ditches are constructed, tending to hasten run-off. Subsoil drainage may, however, increase seepage and tend somewhat to delay run-off. Towns and cities, with paved streets and much relatively impervious surface, as well as drainage systems, tend to quicker and

increased surface flow. The percentage of a drainage area, used for water power, occupied by these artificial works of man is, however, usually small.

3. *Storage.* Natural Lakes and Ponds.—The natural storage of water in lakes and ponds tends to delay run-off, although, unless lakes are very large, the result is chiefly in lessening the peak flow and prolonging the high-water stage. The average yield of a drainage area with much water surface is likely to be low, due to relatively greater evaporation losses from water as compared to land area.

The Great Lakes system affords the most striking example of natural storage, so great in amount as to result in very uniform flow. About one-third of their drainage area consists of water surface, and the monthly

TABLE 27.—HYDROLOGY OF GREAT LAKES. PERIOD 1905–1914, INCLUSIVE
(From letter to the Secretary of War on Diversion of Water from Great Lakes and Niagara River, 1921, pp. 367–368)

Lake	Drainage area			Yearly precipitation, inches	Yearly evaporation from water area, inches
	Total square miles	Water, square miles	Water area per cent of total		
Superior.....	80,700	31,810	39.6	27.6	18.7
Michigan.....	69,040	22,400	32.4	31.7	26.1
Huron.....	72,600	23,010	31.7	29.8	26.1
Erie.....	41,100	10,400	25.4	34.5	35.0
Total or mean at Niagara Falls.....	263,440	87,620	33.3	30.4	25.2
Ontario.....	34,640	7,540	21.7	33.5	33.1
Total or mean, five lakes...	298,080	95,160	32.0	30.7	26.1

Item	At Niagara Falls	At outlet of Lake Ontario (Galops Rapids)
Total land and water area, square miles.....	263,440	298,080
Mean yearly discharge, second-feet.....	205,520	238,350
Mean yearly run-off, inches.....	10.6	10.9
Mean yearly water losses, inches.....	19.8	19.8
Mean ratio, $\frac{\text{run-off}}{\text{precipitation}}$	0.350	0.355

discharge as measured at Buffalo varies only about 20 per cent either way from the mean. On the other hand, the mean run-off is but 35 per cent of the precipitation due to excessive evaporation from the lake surface (see Table 27).

Lakes and ponds are often controlled by outlet works, which may frequently be arranged at moderate cost to raise the high-water level and thus provide a greater storage capacity. By carefully regulating the discharge of stored water, the low-water flow of the stream may be materially bettered and the stream made more valuable for power use.

Artificial Reservoirs.—With storage regulation, artificial reservoirs also tend to make stream flow more uniform, lessening flood peaks and improving the low-water flow. They tend slightly to reduce the average run-off of the stream, however, by introducing new and greater evaporation losses from water area as primarily noted.

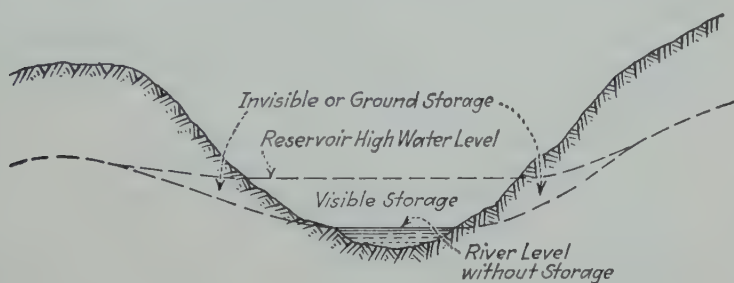


FIG. 21.—Ground storage.

Ground Storage.—The “visible” storage of a reservoir or lake may be supplemented by more or less ground storage in the soil adjacent to its boundaries (see Fig. 21). In sand or gravel this may be considerable in amount and of importance, although difficult to estimate in quantity.

Complexity of Factors Affecting Disposal of Precipitation.—From the foregoing outline and brief discussion it will be seen that run-off is the result of many complex and variable factors more or less interdependent. For illustration, rainfall is caused by the chilling of moist air currents or change of temperature. This may be due directly, however, to topography, as with orographic rainfall. Furthermore, due to the general location of the drainage area, only certain storms may pass over it or it may be (like the Connecticut River) subject to two general classes of storms, usually at separate times but occasionally combined. Again, topography is the result of geologic processes, but the condition of the soil as affecting run-off may be greatly modified by conditions of vegetation or cultivation, as well as seasonal in character, the latter effect being chiefly due to variation in temperature.

It is obviously impossible to determine the effects of these factors separately on a given drainage area, although their consideration is often

of great help in explaining differences in the yield of adjacent streams, as determined by actual measurements, which reflect the composite action of these various factors. The estimate of water losses, which can be to a considerable extent segregated and measured on small areas experimentally, is also of value in such studies.

Water Losses.—As noted, water losses may be classified as follows:

Water area: Evaporation.

Land area: Soil evaporation.

Interception.

Transpiration.

Evaporation from Water Area.¹—Evaporation from water area is the process by which a portion of the water (or ice, in winter) near its surface changes into the gaseous state or water vapor. The molecules of water are in a state of motion, with their velocity dependent largely on temperature, and some of them tend to pass beyond the influence of cohesion and remain above the water as vapor, which may also be returning molecules to the liquid. If the loss and gain to the water surface of these molecules are equal, there is no evaporation; if gain exceeds loss, there is condensation of water vapor.

Stated in another way, at and near the water surface the vapor pressure is that for the saturation point (or maximum vapor pressure) corresponding to the temperature of the water. In the air above the water the actual vapor pressure will depend upon the relative humidity and the temperature, keeping in mind that a relative humidity of 100 per cent (or saturation point) would mean the maximum possible vapor pressure for any given temperature. If the actual vapor pressure of the air above the water is less than that at the water surface (which, as noted above, corresponds to the pressure of saturated vapor at water temperature), evaporation will occur. This fundamental law of evaporation was first stated by Dalton in 1802.

As previously noted, the pressure of saturated water vapor increases rapidly with temperature increase, at ordinary water and air temperatures doubling for approximately every 20° F. rise in temperature, so that temperature is an important factor in causing variation in evaporation. A high water temperature and consequent high vapor pressure at the water surface tend to increase evaporation, while a high air temperature *per se* (for a given relative humidity) tends toward a high vapor pressure in the air over the water and a lessened amount of evaporation. Maximum evaporation from water area, therefore, occurs in summer, after the water has become relatively high in temperature and at times when, with low relative humidity, air temperature is low. Extremely low relative humidity is often more important than air temperature,

¹ The author is indebted to Robert E. Horton, Consulting Engineer, of Albany, N. Y., for much of the material and for his assistance in the treatment of this subject.

however (except as water temperature is thereby affected), as, for illustration, in the desert area of Nevada, where relative humidity is so low as to cause a great difference between the vapor pressure at water surface (which of itself tends to be high in these regions due to high temperatures) and that of the air, resulting in very excessive rates of evaporation.

Wind is also a very important factor in varying the amount of evaporation from water area, tending as it does to remove and break up the blanket of water vapor near and above the water area, carrying it away, lowering the relative humidity of the air over the water and, therefore, its vapor pressure, and thus stimulating further evaporation.

Evaporation on the windward side of a lake is, therefore, greater than on the farther shore, and also greater on the smaller ponds and lakes as far as wind effect is concerned.

Measurement of Evaporation from Water Area.—Evaporation from water surface can be measured directly by exposing a vessel of water and noting by weight or otherwise its change in amount. In 1887-1888 the U. S. Weather Bureau made extensive observations of evaporation throughout the United States, using the Piche evaprometer (rated by comparison with evaporation measured in a vessel of water) and readings of wet- and dry-bulb thermometers.¹ These experiments showed an amount of annual evaporation varying from less than 20 in. in the Northwest and Northeast to over 100 in. in the desert areas of Nevada, Arizona, and California. They were laboratory determinations, however, and, while the results are of some value to give the relative evaporation in different sections of the country, they are obviously not at all applicable to the case of lakes, ponds, and reservoirs.

Several determinations of evaporation from pans floating in reservoirs have been made, however. The method of observation has usually been that of measuring the loss of water in the pan when floating with its top 2 or 3 in. above the water surface of the reservoir. The pan is commonly supported by pontoons and arranged to float inside of a skeleton log raft 10 to 15 ft. square. A pan 3 ft. square and 18 in. deep has been extensively used by the U. S. Geological Survey, although a circular pan avoids error in measurement due to the pan not being level. A spindle with a sharp point is fixed vertically in the middle of the pan, with its point 1 or 2 in. below the top.

In measuring the amount of evaporation, the water surface is made of exactly the same height as the point of the spindle, and, then, at the next time of observation, the process is repeated, the amount of water required to restore the water surface to the level of the spindle being noted. The spindle is surrounded by a thin iron cylinder or stilling box about 3 in. in diameter, with its axis parallel to the spindle and closed with the exception of some small holes near the bottom. This dampens

¹ See *Monthly Weather Rev.*, September, 1888.

movements of the water surface and enables very close determinations to be made of its height. A small cup of such capacity that it represents 0.01 in. depth of water in the pan is used for pouring in the water (or dipping it out in case it has rained and the rainfall has exceeded the

TABLE 28.—EVAPORATION FROM WATER SURFACE, IN INCHES, AT MT. HOPE RESERVOIR, ROCHESTER, N. Y. 1896–1929 INCLUSIVE

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1896	0.45	<u>0.45</u>	<u>0.91</u>	2.72	<u>5.72</u>	6.05	5.19	5.23	3.38	2.45	1.50	1.20	35.25
1897	0.59	0.62	1.76	2.45	3.36	5.01	<u>4.43</u>	5.37	4.66	3.33	1.33	0.96	33.87
1898	0.56	1.01	1.56	<u>3.41</u>	3.64	5.65	<u>6.86</u>	4.98	3.95	2.57	1.57	1.54	37.30
1899	1.53	0.87	2.25	2.96	3.55	5.81	6.40	<u>6.05</u>	4.36	2.59	1.69	1.31	<u>39.37</u>
1900	0.79	1.29	1.11	3.02	<u>4.04</u>	5.83	5.08	<u>4.94</u>	4.58	3.31	1.51	0.98	36.48
1901	0.73	0.60	0.93	2.15	2.82	4.88	5.39	3.87	3.53	2.55	1.24	0.95	29.64
1902	0.44	0.76	1.68	2.79	3.88	3.68	3.64	<u>2.64</u>	3.42	1.88	1.52	1.80	28.13
1903	<u>0.34</u>	0.85	1.38	3.20	5.37	3.00	4.73	<u>4.43</u>	4.12	2.41	1.92	0.97	32.72
1904	0.51	1.10	1.73	1.80	3.34	3.87	4.16	4.86	3.08	2.88	1.40	0.78	29.51
1905	0.87	0.76	1.69	2.26	3.26	2.84	4.26	4.41	3.25	2.72	1.56	1.21	29.09
1906	1.16	0.75	0.80	2.69	3.42	3.98	3.94	4.38	3.93	<u>1.44</u>	1.10	0.74	28.33
1907	1.05	0.39	1.50	1.83	2.97	3.19	4.57	4.91	<u>2.38</u>	2.12	<u>0.91</u>	1.39	27.21
1908	1.11	0.80	2.46	2.82	2.67	4.73	4.97	4.86	3.69	2.60	1.52	1.37	33.60
1909	0.95	1.19	1.50	1.79	2.99	5.02	5.19	5.48	3.43	2.37	1.23	1.12	32.26
1910	1.13	<u>1.89</u>	2.21	2.44	3.70	4.83	5.22	4.00	2.70	2.48	0.97	<u>0.54</u>	32.20
1911	0.48	<u>0.71</u>	1.40	2.56	4.40	4.52	5.82	4.19	3.00	2.05	1.13	1.23	31.49
1912	0.68	0.96	1.33	2.19	2.85	4.91	4.90	3.16	2.46	2.28	1.53	1.44	28.69
1913	<u>1.62</u>	0.89	2.52	2.54	4.07	4.50	5.54	5.38	3.52	2.26	1.52	1.45	35.81
1914	0.56	0.76	1.82	1.46	3.32	4.03	4.61	3.76	3.27	2.07	1.63	1.01	28.30
1915	0.63	0.99	1.17	2.31	3.21	3.79	3.14	3.27	3.09	2.30	1.28	0.85	26.03
1916	1.10	1.14	1.88	1.53	2.70	<u>2.46</u>	4.47	4.32	4.00	2.37	1.39	1.17	28.53
1917	0.78	0.76	1.77	2.21	2.97	2.52	<u>2.76</u>	4.62	3.43	1.99	1.31	1.15	26.27
1918	0.70	0.88	<u>2.80</u>	2.51	2.88	4.19	4.06	4.59	2.85	1.77	<u>2.27</u>	1.07	30.57
1919	0.83	0.77	2.12	1.83	<u>2.05</u>	2.83	3.97	3.19	3.07	<u>4.36</u>	1.47	0.70	27.19
1920	0.54	0.54	1.94	2.41	3.56	3.28	4.05	3.43	3.38	<u>2.28</u>	1.48	1.14	28.03
1921	1.23	0.75	1.46	2.19	3.71	4.73	3.89	4.51	3.78	2.38	1.61	1.63	31.87
1922	0.97	0.62	1.04	2.89	3.23	6.13	4.79	4.22	3.39	2.72	1.26	2.13	33.39
1923	1.54	0.73	2.19	2.68	3.47	<u>6.58</u>	4.49	5.13	2.67	2.92	1.27	1.51	35.18
1924	1.17	1.00	1.57	<u>1.38</u>	2.31	2.84	3.86	3.68	3.19	2.57	2.10	1.59	27.26
1925	1.13	1.06	1.57	2.74	3.18	3.86	3.50	3.93	<u>5.04</u>	4.03	1.30	0.75	32.09
1926	0.36	0.85	1.59	1.52	3.58	3.81	4.93	3.41	2.59	2.25	1.36	0.67	26.92
1927	0.75	1.69	2.29	3.19	5.65	5.71	6.68	4.05	3.55	2.34	1.83	1.01	38.74
1928	0.90	0.66	1.00	2.19	2.37	2.53	3.22	2.82	3.20	1.70	1.74	1.02	23.35
1929	0.85	0.50	1.80	2.70	2.78	2.67	4.60	4.42	3.69	5.70	1.59	1.62	32.92
(34 years)													
Mean.....	0.82	0.87	1.67	2.39	3.44	4.24	4.63	4.31	3.46	2.59	1.47	1.18	31.07
Sum.....	29.03	29.59	56.73	81.36	117.02	144.26	157.31	146.58	117.63	88.04	50.04	40.00	1057.59

(Maxima and minima are underlined.)

evaporation), so that the number of cupfuls represents the change in depth in hundredths of an inch of the evaporation if there has been no rainfall. A rain gage is maintained on the raft so that correction can be made for any rainfall.¹

¹ See U. S. Geol. Survey, *Water Supply Paper* 279, pp. 113–129, for description of such a station and results obtained.

TABLE 29.—MONTHLY EVAPORATION IN INCHES FROM WATER AREA

Month	Chestnut Hill, Mass., 1875-1890			Maine, 1905- 1907			Rochester, N. Y., 1896-1929			Chapel Hill, N. C., 1921- 1924			Charleston, S. C., 1904-1930			Lake of the Woods, 1913- 1915			Lake Tahoe, California, 1900-1906			Santa Clara Valley, California, 1904-1905		
	Mean	Maxi- mum	Mini- mum	Mean	Maxi- mum	Mini- mum	Mean	Maxi- mum	Mini- mum	Mean	Maxi- mum	Mini- mum	Mean	Maxi- mum	Mini- mum	Mean	Maxi- mum	Mini- mum	Mean	Maxi- mum	Mini- mum	Upper Gorge, mean	Laguna Seca, mean	
January.....	0.96			0.7	0.82	0.34	1.37	2.32	3.32	1.01	1.81	0.70	1.11	1.81	0.70	0.3 ±	0.86	1.30	0.47	0.9	0.9	1.0	1.0	
February.....	1.05			0.7	0.87	0.45	1.41	2.65	4.03	0.69	1.30	0.47	0.86	1.30	0.47	0.3 ±	0.86	1.30	0.47	0.9	0.9	1.0	1.0	
March.....	1.70			1.1	1.67	0.91	3.10	3.95	6.35	1.98	1.88	0.56	1.24	1.88	0.56	0.6 ±	1.24	1.88	0.56	2.3	2.3	2.6	2.6	
April.....	2.97	3.12	2.78	1.6	2.39	0.91	3.79	4.94	6.36	2.42	2.75	1.25	1.85	2.75	1.25	1.41	1.85	2.75	1.25	3.1	3.1	3.7	3.7	
May.....	4.46	5.89	3.35	2.1	3.44	1.38	4.22	5.37	7.39	2.83	3.20	1.08	2.44	3.20	1.08	1.61	2.44	3.20	1.08	4.5	4.5	5.2	5.2	
June.....	5.54	7.01	3.94	2.8	4.24	2.05	5.22	5.68	8.65	4.38	3.90	1.39	3.32	3.90	1.39	2.35	3.32	3.90	1.39	6.3	6.3	7.3	7.3	
July.....	5.98	7.09	4.82	4.32	4.63	2.46	5.26	5.29	7.77	3.64	4.42	3.40	4.42	4.42	3.40	2.74	4.42	4.42	3.40	7.7	7.7	8.9	8.9	
August.....	5.50	7.41	4.25	5.00	4.31	2.76	5.13	5.12	7.77	3.13	4.90	3.45	4.90	4.90	3.45	3.46	4.90	4.90	3.45	7.2	7.2	8.3	8.3	
September.....	4.12	5.13	3.08	3.32	3.46	2.38	4.43	4.49	6.44	2.55	3.83	3.10	3.83	3.83	3.10	3.42	3.83	3.83	3.10	6.3	6.3	7.3	7.3	
October.....	3.16	4.13	2.51	2.2	2.59	1.44	2.82	4.22	7.01	2.72	3.09	2.15	3.09	3.82	2.15	2.18	3.09	3.82	2.15	3.1	3.1	3.6	3.6	
November.....	2.25	3.00	0.66	1.3	1.47	0.91	1.61	2.90	4.19	1.78	2.15	1.35	2.15	2.78	1.35	1.92	2.15	2.78	1.35	1.8	1.8	2.1	2.1	
December.....	1.51			0.7	1.18	0.54	1.38	2.24	3.38	1.07	1.83	0.62	1.30	1.83	0.62	0.4 ±	1.30	1.83	0.62	0.9	0.9	1.0	1.0	
Year.....	39.20	48.00	30.61	25.84	31.07	23.35	39.74	49.17	65.57	34.23	20.7	30.51	40.15	20.52	45.0	52.0	30.51	40.15	20.52	45.0	52.0	52.0	52.0	
Mean temperature.....	48.6			41.0	47.3		61.1	65.6		34	43.9		43.9				43.9			58.1				
Mean precipitation.....	44.70			40.0	33.4		49.29	47.80		22	27.12		27.12				27.12			16.56				

NOTE: Maine records are a mean at four stations in central and northern Maine by U. S. Geol. Survey, "Temperature and Precipitation at Millinocket," *Water Supply Paper* 267.

Rochester records at Mt. Hope reservoir.

Chapel Hill records by North Carolina Geological and Economic Survey.

Charleston records by Water Department (Commissioners of Public Works).

Lake of the Woods records are at Keewatin, Ont., International Joint Commission (1917) report.

Lake Tahoe is about 12 miles from Truckee, at which temperature is given.

Santa Clara Valley figures are for two reservoirs, Laguna Seca and Upper Gorge. Results are "safe figures." Temperature at San Jose, 12 miles away.

The last two sets of records are by the U. S. Reclamation Service (*Eng. News*, Feb. 29, 1912, p. 380).

During the frozen season, evaporation is usually measured by filling a dish, allowing it to freeze solidly, and then exposing it. The loss by evaporation is determined by weighing. Winter observations of evaporation are liable to interruption from rain, sleet, or snow. Moreover, results observed in this way are probably not closely comparable with actual evaporation from a large frozen reservoir. On the other hand, the total evaporation during the frozen season is only perhaps 10 per cent of that for the year, and errors in its measurement are not, therefore, of great importance.

Amount of Evaporation from Water Surface.—Two long and carefully observed sets of evaporation records from floating pans have been kept in this country, *viz.*, (1) at Chestnut Hill Reservoir, Boston, from 1875 to 1890, by Fitzgerald,¹ and (2) at Mt. Hope Reservoir, Rochester, N. Y., from 1891 to 1929 (see Table 28) under the initial direction of Kuichling and later the City of Rochester Board of Public Works. Records covering a few years time have also been obtained by the U. S. Geological Survey and Reclamation Service at several different points.² Some of these records are summarized in Table 29.

It will be noted that in general there is not so great a range in the amount of evaporation for any given month of the year as of precipitation. Hence, a shorter time is required to obtain data of average monthly evaporation sufficiently accurate to use in storage estimates.

Another rough average relation established from the table by plotting annual evaporation against mean temperature is that yearly evaporation increases approximately at the rate of about 1 in. for each degree Fahrenheit increase in temperature, or about as follows:

Temperature, Degrees Fahrenheit	Annual Evaporation, Inches
30	15
40	25
50	35
60	45

It must be kept in mind, however, that such a relation between temperature and evaporation ignores the other factors which largely affect evaporation, *viz.*, relative humidity and wind, and, hence, is at best but a rough approximation.

Limitations in Use of Evaporation Data.—Measurements of evaporation by the floating-pan method do not usually correctly represent the evaporation rate from the reservoir or lake surface. Water temperature in the pan usually differs from that in the lake, and wind action is modified

¹ See *Trans. A.S.C.E.*, 1886, p. 581.

² See also Houk, I. E., "Evaporation on U. S. Reclamation Projects," *Proc. A.S.C.E.*, January, 1926.

by the projecting rim of the floating pan. Furthermore, the "vapor blanket" which forms on the lake tends to lessen actual, as compared with pan, evaporation. Hence, measured pan evaporation is usually considerably greater than actual evaporation, and the latter must be obtained by applying an "area factor" to the former, which will commonly be between 0.5 and 0.8.

Evaporation Principles.—Some of the more important results of theory and experience in regard to evaporation are as follows: Three processes are involved in evaporation, *viz.*: (a) vapor emission, (b) vapor removal, and (c) vapor return.

a. Vapor emission from water is directly proportional to the maximum vapor pressure corresponding to water temperature.

b. Vapor removal takes place by the three processes of diffusion, convection, and wind action. Diffusion continues whether or not wind action occurs, but convection does not ordinarily occur when there is wind action.

The rate of vapor removal by wind action increases with wind velocity but not in direct proportion, the maximum possible effect occurring when all the newly emitted vapor is removed by the wind. The effect of wind action varies with relative humidity, in perfectly dry air the maximum effect of wind action as determined by Browne and Escombe¹ being to double the evaporation rate as compared with that of still air.

c. Vapor Return.—In still air evaporation equilibrium occurs when the actual vapor pressure V in the air equals the maximum vapor pressure V_w , corresponding to water-surface temperature. If V is greater than V_w , vapor return or condensation occurs in still air.

If air and water temperatures are equal, the maximum vapor pressure in the vapor blanket V_b will be equal to V_w (the maximum vapor pressure at water temperature), which will equal V_a (the maximum vapor pressure corresponding to air temperature). In still air evaporation would then cease, but wind would lower V_b by vapor removal through mechanical action so that evaporation may continue.

When the air is warmer than water, the air near the water surface is cooled, tending to form a layer of heavy, stable air near the water surface with temperature intermediate between that of air and water. Under these conditions, unless prevented by wind, V_b can build up to a value greater than V_w and approach V_a as a limiting value.

With air colder than water the layer of air close to the water surface is warmed and the vapor pressure therein increased by vapor emission. The air near the water surface will, therefore, become lighter than that above, producing instability and convective vapor removal, which may often be seen in the form of columnar vapor drift over a lake on an autumn morning when the water surface is warmer than the air. In this case evaporation continues even though the overlying air is saturated, when,

¹ LLOYD, E. E., "Physiology of Stomata," Carnegie Inst., *Pub.* 82.

however, visible condensation also results and vapor return occurs, both as true vapor and liquid droplets.

Evaporation Formulas.—Various evaporation formulas have been deduced or suggested, some of which give good results under ordinary conditions but fail for limiting conditions such as maximum wind velocity, very dry air, or high relative humidity.

Fitzgerald¹ developed a formula based upon his actual pan measurements, consistent with the Dalton law, as follows:

$$E = C(V_w - V)(1 + KW) \quad (1)$$

where C is a numerical factor varying with the time period considered, W is the wind velocity, and K a wind coefficient. By this formula, therefore, evaporation increases indefinitely as wind velocity increases.

Meyer's modified Dalton formula² is in the same form:

$$E_m = 15(V_w - V)\left(1 + \frac{W}{10}\right) \quad (2)$$

this giving evaporation in inches per month, W being expressed in miles per hour.

Bigelow's formula³ expressed in English units and with similar notation to that previously used is

$$E_m = 75.8 \frac{V_w}{V} \frac{dV_w}{dT} \left(1 + \frac{W}{9}\right), \quad (3)$$

where dV_w/dT is the rate of change with temperature of maximum vapor pressure at the temperature of water surface. His formula is the result of elaborate studies, based upon floating and fixed pan observations of evaporation at and near the Salton Basin in California, as well as several other points in the United States, made during 1907–1910 by the U. S. Weather Bureau, supplemented by other observations made by the Argentine Meteorological Office. His formula agrees with the Dalton formula when V is small in comparison with V_w , but diverges in humid climates when air temperature is low, as at night.

Bigelow studied the effect of size of water area on evaporation, finding that for a very small area, where the vapor evaporated is instantly removed, the coefficient 75.8 would be practically doubled. His formula as given above is for

. . . a body of water large enough to be fully covered with all its natural vapor, in spite of the wind action, which can only transport the vapor along the surface but cannot remove it, . . . this being the natural maximum rate of evaporation for lakes or reservoirs under all the prevailing wind conditions.

¹ *Trans. A.S.C.E.*, vol. 15, p. 581, 1886.

² MEYER'S "Hydrology," p. 238.

³ Argentine Meteorological Office, *Bull.* 2, 1912.

It is of interest also to note that the actual evaporation from the Salton Sea, based upon the lowering of water level during 1906-1910, checks with the computed amount by the Bigelow formula.

Horton¹ has more recently proposed the formula

$$E = C(\psi V_w - V) \quad (4)$$

where E is evaporation in inches per 24 hr., $C = 0.4$, and ψ is a wind factor expressed by the formula

$$\psi = 2 - e^{-0.2w_0} \quad (5)$$

where w_0 is the wind velocity in miles per hour at ground or water-surface level. Horton gives a curve showing values of ψ from which Table 30 was prepared:

TABLE 30.—WIND FACTOR ψ IN HORTON'S EVAPORATION FORMULA

Wind Velocity, Miles per Hour	Values of ψ	Wind Velocity, Miles per Hour	Values of ψ
0	1.0	12	1.91
1	1.18	13	1.92
2	1.34	14	1.94
3	1.46	15	1.95
4	1.56	16	1.96
5	1.63	17	1.97
6	1.70	18	1.97
7	1.75	19	1.98
8	1.80	20	1.98
9	1.83	25	1.99
10	1.86	30	2.00
11	1.89		

To get w_0 Horton has also deduced the following formula intended for use ordinarily in the East, based on an observed w_h at height h above the ground:

$$w_h = a + b\sqrt{h + c} \quad (6)$$

where

$$\begin{aligned} a &= \frac{2}{3}w_0 \\ b &= \frac{w_0 - a}{\sqrt{c}} \\ c &= \frac{30}{w_0} \end{aligned}$$

For convenience in use the relation of w_0 and w_h by Eq. (6) is shown on Fig. 22.

When there is no wind but the water surface is warmer than the air (as often at night), vapor removal occurs mainly by convection, the effect of which Horton obtains by using for w in the previous table the

¹ *Eng. News-Record*, Apr. 26, 1917, pp. 196-199.

quantity $\sqrt{\theta - \theta_a}$ where θ and θ_a are temperatures of water surface and air, respectively.

Computed Evaporation from Water Area.—The essentials for computing evaporation by any of the foregoing formulas are (1) temperature of water, (2) temperature of air, (3) relative humidity, (4) wind velocity. These are all factors which can readily be measured with reference to any reservoir, and such a record will enable a fair approximation of evaporation to be made, at least for monthly periods. Rough values can also be obtained by assuming water and air temperatures the same, where observations of water temperature are lacking. In warm climates these temperatures do not differ greatly, but in the North there is a considerable lag of water temperature behind that of the air as the seasons change, so that the spring months (with water colder than air) tend to show less

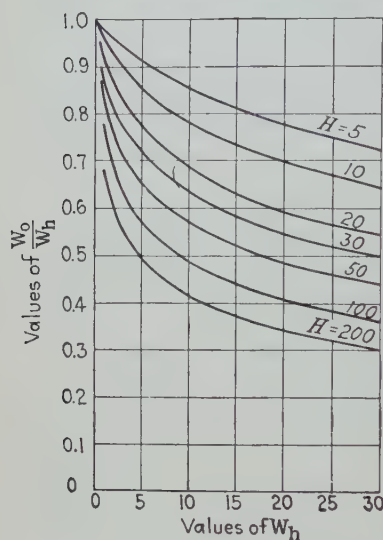


FIG. 22.

evaporation for any given air temperature than the fall months (when the air is cooler than the water).

Examples of Computation of Evaporation.

Assumed data (as observed by R. E. Horton) month of July, 1923, Voorheesville near Albany, N. Y.:

Air temperature.....	67° 6
Water temperature.....	72° 6
Wind velocity.....	1.05 miles per hour
Observed evaporation.....	5.99 in. from standard evaporation pan

U. S. Weather Bureau data at Albany for July, 1923:

Air temperature.....	70° 6
Wind velocity (115 ft. above ground)	6.3 miles per hour
Relative humidity.....	0.623

By Dalton-Meyer formula:

$$E = 15(0.799 - 0.420) \left(1 + \frac{1.05}{10} \right) = 6.28 \text{ in.}$$

By Bigelow formula:

$$E = 75.8 \times \frac{0.799}{0.445} \times 0.027 \left(1 + \frac{1.05}{9} \right) = 4.15 \text{ in.}$$

By Horton formula:

$$\psi = 1.19, \quad C = 0.4 \times 30.4 = 12.2$$

$$E = 12.2(1.19 \times 0.799 - 0.420) = 6.35 \text{ in.}$$

The above results are the computed values of pan evaporation by the several formulas. As will be noted, both the Dalton-Meyer and the Horton formulas agree closely with the measured pan evaporation.

To estimate the corresponding evaporation from reservoir area requires the application of the "area factor" F , which may be computed as follows by Horton's method.

Area Factor.—The area factor is defined as the ratio of lake or reservoir evaporation to that from a measuring pan. It will, therefore, be

$$F = \frac{C(\psi V_w - V_b)}{C(\psi V_w - V)} \quad (7)$$

where V_b is the maximum vapor pressure within the vapor blanket immediately over the lake surface.

If air and water temperatures are equal, $V_w = V_b$ and $V = hV_w$, where h = relative humidity = V/V_a .

Hence, for this case

$$F = \frac{\psi - 1}{\psi - h} \quad (8)$$

If air and water temperatures are not equal, the area factor will be

$$F = \frac{\psi - r}{\psi - h^1} \quad (9)$$

where

$$r = \frac{V_b}{V_w} \text{ and } h^1 = \frac{V}{V_w}.$$

Thus, with equal air and water temperatures, F will vary as shown in Table 31 for a number of different locations.

TABLE 31.—EVAPORATION-AREA FACTOR F , FOR DIFFERENT LOCALITIES
(Air and Water Temperatures Alike)

Station	w_0	ψ	h	F		
				Year	Summer	Winter
Boston, Mass.....	4.5	1.59	0.72	0.68	0.68	0.68
Albany, N. Y.....	4.4	1.58	0.76	0.71	0.69	0.73
Harrisburg, Pa.....	3.8	1.53	0.72	0.66	0.64	0.68
Charleston, S. C.....	5.3	1.65	0.78	0.75	0.75	0.74
Chicago, Ill.....	5.2	1.65	0.75	0.72	0.68	0.76
Bismarck, N. D.....	5.6	1.67	0.69	0.68	0.66	0.70
Helena, Mont.....	4.5	1.59	0.56	0.58	0.54	0.61
Salt Lake, Utah.....	2.8	1.43	0.53	0.49	0.43	0.55
Santa Fe, N. M.....	4.9	1.62	0.45	0.55	0.53	0.57
San Francisco, Calif.....	3.7	1.52	0.80	0.72	0.75	0.69
Portland, Ore.....	3.5	1.31	0.75	0.67	0.62	0.73

It will be seen from the above table that the area factor F will vary from about 0.4 to 0.75 as extreme limits. A table similar to the above, by months, of the two periods of the year shows variations commonly of only a few per cent as regards mean values for the periods. Hence, the table gives a very good idea of range in mean values of F .

Use of Area Factor.—In the previous example using the U. S. Weather Bureau data for July, 1923, at Albany, as more nearly representative of reservoir conditions:

$$w_h = 6.3$$

whence

$$w_o = 3.35, \text{ and } \psi = 1.48.$$

Then

$$F = \frac{1.48 - 1}{1.48 - 0.62} = 0.56$$

which applied to the computed pan value of 6.16 in. gives 3.45 in. as the corresponding amount of monthly evaporation from reservoir surface.

This is rather low for the month of July, due probably to the fact that the Weather Bureau observations of wind velocity are somewhat affected by instrument enclosure and lower than the true velocity at the station elevation. The application of the area factor directly to the pan measurements of evaporation at the Weather Bureau class A stations is, therefore, likely to give too small results. Probably an arbitrary area factor of about 0.90 is nearer correct in this instance.

Soil Evaporation.—Soil evaporation refers to water losses due to evaporation of rainfall (or snow) as it lies on or just below the ground surface during or subsequent to its occurrence. The same general laws of evaporation apply as with a free-water surface. On the other hand, the condition of the land surface—whether cultivated, in grass, or vegetal cover—the character of soil and subsoil, etc., are of great importance in affecting the amount of such evaporation, in addition to the meteorological conditions of temperature, relative humidity, and wind velocity, which are of fundamental importance. Furthermore, soil evaporation also depends on what Horton¹ has called the “evaporation opportunity,” viz., the time during which it may go on, which varies greatly with the amount and time distribution of the precipitation. A light rain falling upon dry ground, which is often higher in temperature than the air, may evaporate at a rate greater than from a free-water surface. If the rain continues, however, and the ground becomes saturated and approximates air temperature, the loss by evaporation will be similar in amount to that from a shallow-water area. After the rain, as evaporation proceeds, the surface water (on earth and vegetation) soon disappears and the rate of evaporation lessens, although capillary action will continue for some

¹ *Trans. A.S.C.E.*, vol. 79, p. 1171.

time to draw moisture to the surface, so that soil evaporation is more or less continuous, depending upon the frequency of rainfall, although quite variable in rate.

Vegetation, including forests, by shading the ground tends toward a lower temperature as well as slightly greater relative humidity and, hence, a lessened amount of soil evaporation.

Experiments carried on at the University of Maine 1889-1892¹ showed an excess of relative humidity in the forest above that in the open, as follows:

	Per Cent
7 A.M.....	6
1 P.M.....	14
7 P.M.....	10
Mean.....	10

For soil evaporation alone Meyer² gives the following relative values for different conditions of culture and vegetation:

Bare ground.....	1.00
Grain fields.....	0.80
Grass land.....	0.70
Light forests.....	0.60
Dense forests.....	0.20-0.40

The other water losses of transpiration and interception must be also taken into account, however, in studying the net effect of vegetation and forests on water losses, as will be further discussed.

Percolation will be favored by conditions which tend to reduce soil evaporation, *viz.*, a slow, steady precipitation, pervious soil, and flat slopes. Cultivation is also important in increasing percolation and, hence, lessening both soil evaporation and surface flow.

Slope of land surface also materially affects soil evaporation. For the same exposure to air and sun, the evaporation rate will not be affected by slope, but, as the water does not stand on the ground so long on steep slopes as on flat lands, the evaporation opportunity is reduced and, hence, the amount evaporated as well.

The extent of capillary lift and resulting evaporation was investigated by King³ and found to be as shown in table at top of page 90, using cylinders of sand in approximately natural proportions, of sizes from No. 20 to 100 sieves.

Judging from these results, if the ground-water table is 4 ft. or more below the surface, evaporation would be very slight.

¹ RAFTER, GEORGE W., "Natural and Artificial Forest Reservoirs of the State of New York," p. 436.

² "Hydrology," p. 244.

³ U. S. Geol. Survey, *Nineteenth Report*, Part II.

Capillary Lift in Inches above Ground-water Table	Evaporation, Inches per Month
6	3.42
12	3.34
18	2.39
24	1.04
30	0.58

Measurements of soil evaporation have been made experimentally by means of the lysimeter. As used by Graves¹ at Rothamstead, England, this consisted of a strong, open-topped, water-tight slate box or tank set in the ground, with an area of 1 sq. yd. and 36 in. in depth filled with the sodded soil or sand for which he determined the evaporation losses. The rim of the box projected a little above the ground surface so as to cut off any adjacent surface run-off. Rain falling on the lysimeter was either evaporated or passed through as percolation, the latter being drawn off from the bottom of the lysimeter into a measuring vessel in a sunken adjacent pit. In the case of sand, the difference between rainfall as measured by an adjacent rain gage and the amount of percolation represented soil evaporation. Where sodded soil was used, the water losses included some transpiration and interception losses as well.

Following is a summary of results of Graves' experiments:

SUMMARY—GRAVES' LYSIMETER MEASUREMENTS—ROTHAMSTEAD, ENGLAND

Period covered..... 1860-1873 inclusive

Rainfall, inches per year:

Mean.....	25.72
Maximum.....	37.17 (1872)
Minimum.....	15.89 (1864)

Evaporation, inches per year:

Ground (sodded):

Mean.....	18.14
Maximum.....	25.14 (1872)
Minimum.....	12.07 (1864)

Sand:

Mean.....	4.31
Maximum.....	9.10 (1860)
Minimum.....	1.70 (1870)

Water surface:

Mean.....	20.61
Maximum.....	26.93 (1868)
Minimum.....	17.33 (1862)

Ebermayer² made some extensive experiments in forest hydrology in Bavaria, 1868-1871, including lysimeter measurements for different depths of soil, both in the open and in forest. A summary of the princi-

¹ *Proc. Inst. Civil Eng.*, vol. 45.

² "Influence of Forests on Air and Soil," Germany, 1873, translated by R. E. Horton, Michigan Engineer, 1909.

pal results of his experiments is given in Table 32, from which the following appears from mean results:

Evaporation from water area in the forest was only about 37 per cent of that in the open.

In the open, water losses as measured (with 4-ft. depth of soil) were

$$33.95 - 19.39 = 14.56 \text{ in.}$$

In the forest they were

$$26.43 - 15.18 + 7.52 = 18.77 \text{ in.}$$

or not greatly different in amount, the lesser soil evaporation in the forest being offset by the forest interception losses. Note that in neither case are transpiration losses included.

TABLE 32.—EBERMAYER'S EXPERIMENTS ON FOREST HYDROLOGY—SUMMARY—
MARCH, 1868 TO FEBRUARY, 1869 (*R. E. Horton*)

Item	Station					
	Seeshaupt	Ebrach	Rohrbrunn	Johannes- kreuz	Altenfurth	Mean
Soil.....	Calcareous clay	Sandy loam	Sandy loam	Fine sand	Sand and moss	
Forest.....	40-year fir	50-year pine	60-year beech close	Same	Wild pine close	
Precipitation, inches:						
Open.....	34.96	26.82	43.02	39.90	25.07	33.95
Forest.....	25.51	22.45	35.90	30.16	18.13	26.43
Difference.....	9.45	4.37	7.12	9.74	6.94	7.52
Evaporation from water area, inches:						
Open.....	19.55	27.28	26.40	23.45	20.97	22.77
Forest.....	4.03	10.97	7.42	10.87	9.29	8.52
Percolation, inches:						
In open without litter:						
1 ft.....	18.94		28.88	20.73	11.43	18.45
2 ft.....	15.49		29.12	13.10	14.54	16.90
4 ft.....	10.28		28.12	25.14	14.03	19.39
In forest with litter:						
1 ft.....	22.26	17.55	27.62	12.92	11.34	18.34
1 ft.....	26.68	19.53	31.45	11.73	12.59	20.40
2 ft.....	16.90	22.21	30.51	26.48	11.89	21.60
4 ft.....	13.49	12.05	25.93		9.25	15.18
Soil evaporation, inches:						
In open without litter:						
1 ft.....	16.02		14.14	19.17	13.64	14.80
2 ft.....	19.47		13.90	26.80	10.53	16.17
4 ft.....	24.68		14.90	14.76	11.04	16.35
In forest with litter:						
1 ft.....	3.25	4.90	8.28	17.24	6.79	8.09
1 ft.....	-1.17	2.92	4.45	18.43	5.54	6.50
2 ft.....	8.61	0.24	5.39	3.68	6.24	4.83
4 ft.....	12.02	10.40	9.97		8.88	10.32

At Geneva, N. Y.,¹ lysimeter experiments from 1882 to 1887 furnished data of evaporation from sod, bare soil, and cultivated soil. Results by months are appended, showing the following mean annual values:

	Inches
Rainfall.....	23.72
Evaporation:	
Sod.....	20.26
Bare soil.....	16.77
Cultivated soil.....	15.16

Many other lysimeter experiments have been made, mostly in England and Germany and a few in the United States. A tabular summary of general results of such experiments is given on pages 19 and 23 of *Memoir* 12, June, 1918, of the Cornell University Agricultural Experiment Station, by Lyon and Bizzell. Experiments at Cornell, covering 5 years (1910–1915) on both cropped and bare soil, are also summarized on pages 16 and 17 of this same bulletin, as follows:

	Inches
Rainfall, mean.....	31.14
Evaporation:	
Bare soil.....	6.74
Cropped soil.....	14.18

The soil used was 4 ft. deep, classified as Dunkirk clay loam, heavy and compact and free from stones.

Figure 23 shows a cross section of the lysimeters used by Lyon and Bizzell. There were 24 of the tanks in two rows, built of concrete, each 4 ft. 2 in. square and about 4 ft. deep. Between the rows a tunnel was provided, with receptacles to catch the drainage water.

The great variation in the amount of soil evaporation under different conditions is apparent from the few examples of lysimeter experiments given above, which shows a range for bare soil or sand of yearly amounts averaging from about 4 to 17 in. Many of these experiments include other water losses of transpiration and interception, as noted, which cannot be segregated where vegetation, crops, or sod are existent.

For approximating soil evaporation alone, Meyer² gives a convenient diagram, shown in Fig. 24, which takes into account temperatures as well as evaporation opportunity as shown by rainfall. He states further that actual evaporation may require a coefficient from 0.95 to 1.25 for drainage areas in the Northwest, and similar ones elsewhere. Extreme values of this coefficient he gives as from about 0.60 to 1.25. The use of this diagram will be obvious.

Interception.—Interception refers to water losses due to evaporation of rainfall (or snow) caught and held in suspension by vegetation. Hor-

¹ *Annual Reports*, N. Y. Agr. Expt. Sta. at Geneva, 1883–1890, inclusive.

² *Trans. A.S.C.E.*, 1915, p. 1099.

ton¹ has made an excellent summary of the information available on this subject, as well as some experimental measurements, which article has been largely used as a basis for the following discussion.

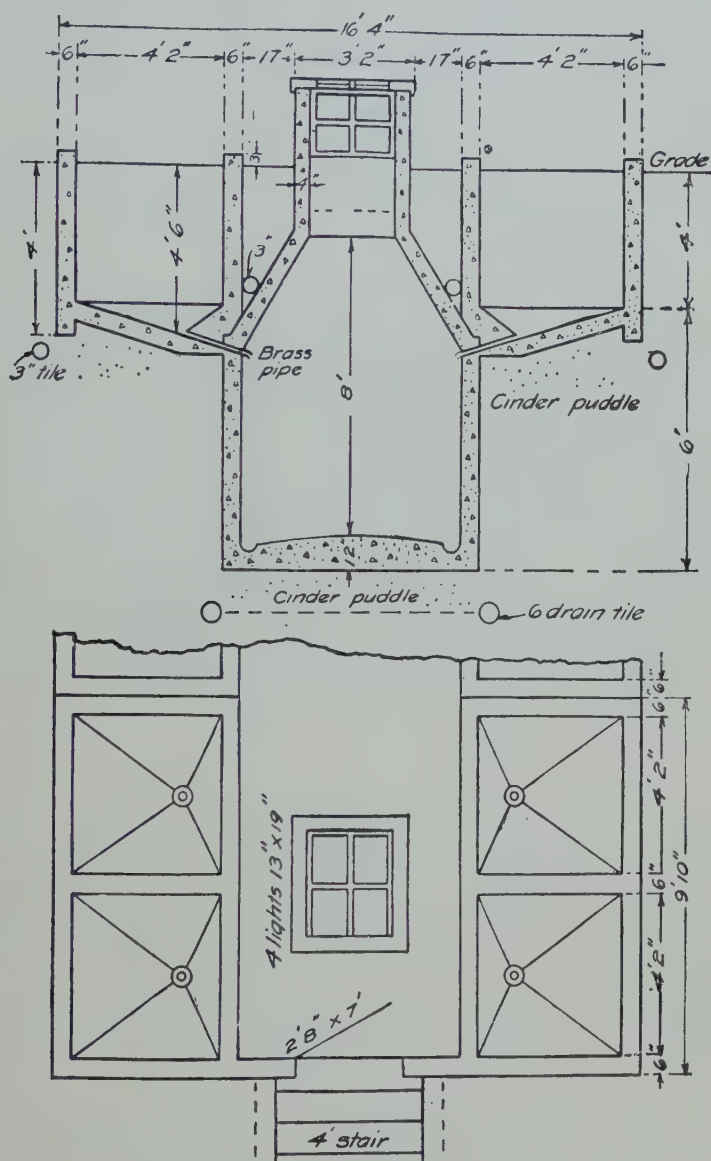


FIG. 23.—Lysimeters used by Lyon and Bizzell.

The percentage of precipitation reaching the ground in forest or on fields with growing crops is very small in the earliest stages of a rain,

¹ HORTON, R. E., "Rainfall Interception," *Monthly Weather Rev.*, September, 1919.

increasing as the duration of the storm increases, the total amount reaching the ground being small for short, light showers and increasing for severe prolonged storms.

It is evident that the amount of interception in a given shower comprises two elements. The first may be called interception storage. If the shower continues and its volume is sufficient, the leaves and branches will reach a state where no more water can be stored on their surfaces. Thereafter, if there is no wind, the rain would drop off as fast as it fell, were it not for the fact that even during rain there is a considerable evaporation loss from the enormous wet surface exposed by the tree and its foliage. As long as this evaporation loss continues and after the

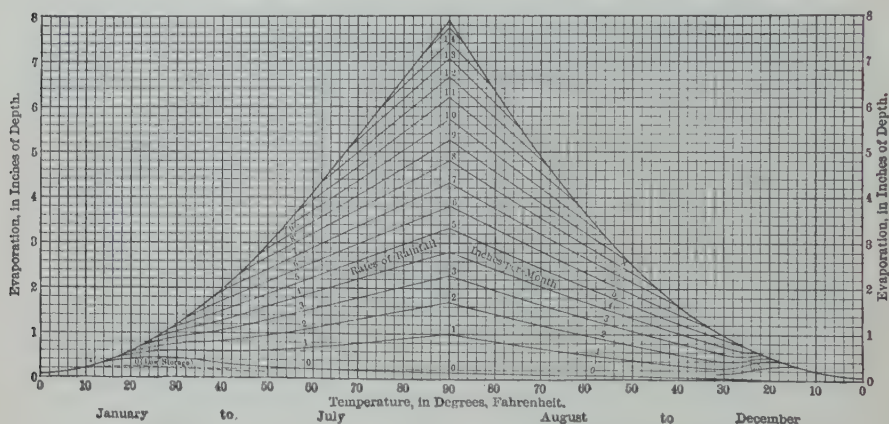


FIG. 24.—Meyer soil-evaporation curves.

interception storage is filled, the amount of rain reaching the ground is measured by the difference between the rate of rainfall and the evaporation loss. When the rain ceases, the interception storage still remains on the tree and is subsequently lost by evaporation. If there is wind accompanying the rain, then, owing to motion of the leaves and branches, it is probable that the maximum interception-storage capacity for the given tree is materially reduced as compared with still-air conditions. Furthermore, in such a case, after the rain has ceased, a part of the interception storage remaining on the tree may be shaken off by the wind, and the storage loss in such a case is measured only by the portion of the interception storage which is lost by evaporation and is not shaken off the tree after the rain has ceased. One effect of wind is, therefore, to reduce materially the interception storage.

The effect of wind is to increase materially the evaporation during a rain so that, while the depth of interception during the early part of the storm is likely to be less than for a storm without wind, the total intercep-

tion depth for a long-continued storm with wind may be the greater of the two, owing to increased evaporation.

Owing to the great extent of leaf surface, the evaporation loss from leaf surfaces is much greater than from the projected area shaded by the tree but is likely to be relatively small per unit of exposed surface compared with the evaporation rate in fair weather, owing to the higher relative humidity common during rain, and the approximate equality between temperature of leaf surface and air.

In general, total interception equals leaf or plant storage capacity plus evaporation loss during a storm, and, as the storage capacity is approximately constant at a given stage of growth, it is evident that the percentage of the rainfall lost by interception tends to decrease with the duration and intensity of the storm. This is confirmed by such experimental data as are available, although the older experiments dealt more particularly with monthly and seasonal interception losses.

Measurements of interception have been made in a number of cases by exposing rain gages under the vegetation or trees and comparing the amount of rain thus collected with that in similar gages, either above or at one side of the vegetation, in the open.

Ebermayer in his studies of forest meteorology previously mentioned found as an average for five stations a rainfall of 33.95 in. in the open and 26.43 in. in the forest; an annual interception loss of 7.52 in., or about 22 per cent of the rainfall, about equally divided between summer and winter and no great difference between evergreen and beech trees.

Horton in his experiments near Albany, N. Y., during 1917 to 1918 endeavored to ascertain the interception loss per storm, as there is generally a fairly close correlation between shower duration and amount of rainfall. Some of his conclusions were as follows:

The interception-storage loss for trees varies from 0.02 to 0.07 in. per shower and approaches these values for well-developed crops.

The interception-storage loss for trees in woods is greater, but the evaporation loss during rain is less than for trees in the open.

The percentage of total precipitation loss is greater in light than in heavy showers, ranging from nearly 100 per cent where the total rainfall does not exceed the interception-storage capacity to about 25 per cent as an average constant rate for most trees in heavy rains of long duration.

Light showers are much more frequent than heavy ones, and the interception loss for a given precipitation in a month or season varies largely, according to the rainfall distribution.

Expressing the interception loss in terms of depth on the horizontal projected area shadowed by the vegetation, the loss per shower of a given amount is very nearly the same for various broad-leaved trees during the summer season.

The amount of water reaching the ground by running down the trunks of trees may amount to a relatively large volume when measured in gallons for a smooth-bark tree in a long heavy rain. It is, however, a relatively small percentage, commonly 1 to 5 per cent, of the total precipitation. The percentage increases from zero in light showers to a maximum constant percentage in heavy showers of long duration.

The interception loss from needle-leaved trees, such as pines and hemlocks, is greater both as regards interception storage and evaporation during rain than from broad-leaved trees.

The average duration of showers of a given intensity is greatest in winter and the colder months, and least in midsummer or thunderstorm months, whereas the evaporation rate is greatest in midsummer and least in the colder months. As a result of the opposite effects of these two factors affecting interception loss, the average loss per shower of a given intensity seems to be nearly constant throughout the different months of the summer period, May to October, inclusive.

Data are insufficient for a final determination of the relative losses from trees in winter and in summer. Apparently the winter and summer losses for a given monthly precipitation for needle-leaved trees are about equal, whereas for deciduous, broad-leaved trees the winter interception loss appears to be about 50 per cent as great when the trees are defoliated as during the growing season.

Interception loss from full-grown field crops approaches in value that from trees, but, owing to the short time during which crops stand on the ground in a fully developed stage of growth, the total annual interception loss from cropped areas is very much smaller than from wooded areas.

The average interception loss from 11 trees, excluding peripheral interceptometers and excluding hickory, for which the results are defective, during the summer of 1918 was 40 per cent of the precipitation.

Horton's mean curve of interception losses per shower for various trees gives values as follows:

Precipitation in Shower, Inches	Interception Losses, Per Cent
0.04	100
0.10	50
0.20	37
0.40	30
0.60	26
0.70 and over	25

In Fig. 25 is shown an interceptometer pan as used by Horton under an elm tree, in his measurements of 1917-1918. There are also shown the lead trough and pail attached to the tree used for catching and measuring the water running down the tree trunk. The bark of the tree was first smoothed, then the sheet lead calked into the bark and the joint made water-tight with paraffin.

Horton also (Table 21 of his article) gives an example of calculated interception losses for the drainage area of the Seneca River, above Seneca Falls, N. Y. (a farming country), by months from May to October, inclusive, 1914, which total 2.23 in. depth for the 6 months' period, during which time the precipitation was 20.04 in., or an average inter-



FIG. 25.—Interceptometer under elm tree, as used by Horton.

ception loss of about 11 per cent of the precipitation. The distribution of losses is shown in Table 33:

TABLE 33.—CALCULATED INTERCEPTION LOSSES BY HORTON—SENECA RIVER DRAINAGE, MAY TO OCTOBER, 1914

Crop or character of subdivision	Per cent of land area	Interception losses		
		Amount, inches	Amount, inches on whole area	Per cent of total
Meadow.....	19.5	1.82	0.372	16.7
Pasture.....	21.5	1.10	0.238	10.7
Wheat, rye, barley.....	5.7	2.35	0.156	7.0
Oats.....	6.7	1.69	0.129	5.8
Potatoes, etc.....	8.3	0.80	0.064	2.9
Corn.....	4.4	1.00	0.044	2.0
Buckwheat.....	5.8	1.87	0.050	2.2
Orchards and vineyards.....	5.0	4.27	0.349	15.7
Woods.....	10.0	8.27	0.826	37.0
Roads, etc.....	5.0	0.0	0.000	0.0
	91.9		2.228	100.0

In the foregoing table newly plowed areas, upon which there would be no interception losses, are excluded. As will be noted, the loss per unit of area is less from crop land than from wooded land, due chiefly to the fact that crops are at their full stage of development as a rule only 1 to 3 months per year, while the interception capacity remains nearly constant during the summer season.

Some hydrologists have included interception losses with those of transpiration. It is quite evident, however, that their laws of variation differ greatly, and, as at times interception losses amount to one-third or more of the rainfall, it is often important to consider their effect separately in studies of hydrology.

Transpiration.—Transpiration refers to the vaporization of water from the breathing pores of leaves and other vegetable surfaces. It is a complex phenomenon depending upon the amount and distribution of rainfall, as well as the meteorological conditions which affect evaporation, *viz.*, temperature, humidity, and wind velocity. The yield of crops or the growth of vegetation is a measure of the effect of these factors on vegetative activity; hence, the amount of transpiration is closely allied to the vegetative activity. Light is also an important factor in transpiration, the greater portion of this occurring during the daytime, as shown by various experiments.

Meyer¹ has concluded that transpiration varies approximately with temperature, according to Van't Hoff's law that most chemical reactions and physiological processes double in activity for every increase of about 18° F. in temperature. He states that this law has been found experimentally to be substantially correct for the rate of fixation of CO₂ by plants in sunlight, and, as transpiration occurs during this process, its rate, as far as dependent on temperature, would follow the same law. His "base curve" of transpiration, starting at 40° F. in spring and ending at 43° in November (below which temperatures vegetable cells are assumed to be inactive), departs but little from the relations:

$$\left. \begin{array}{l} T_M = 0.085(t - 40) \text{ (March to July)} \\ \text{and } T_M = 0.090(t - 43) \text{ (August to November)} \end{array} \right\} \text{up to } 85^\circ \text{ F.} \quad (10)$$

where T_M = monthly transpiration in inches depth.
 t = monthly air temperature in degrees Fahrenheit.

Above a temperature of 85° F. is assumed a period of "summer rest" for plants.

Such an assumed law of transpiration takes into account only temperature, whereas the available moisture supply, character of vegetation, hours of sunshine, etc., all may be important. Moreover, the monthly transpiration varies materially from year to year even with similar forms of vegetation. Meyer, therefore, suggests that his mean curve is to be used merely as one of the guides in arriving at the monthly distribution of a given quantity of seasonal transpiration. He gives as normal

¹ *Trans. A.S.C.E.*, 1915, p. 1090.

seasonal transpiration (which also includes interception losses) for the north central United States:

- 9 to 10 in. for grains, grass and crops.
- 8 to 12 in. for deciduous trees.
- 6 to 8 in. for small trees and brush.
- 4 to 6 in. for coniferous trees.

In any particular month the transpiration amount based on his "mean curve" is to be modified for deficient or excess rainfall as well as available ground-water supply.

Measurements of transpiration as a segregated water loss have been made, particularly for growing crops, by several experimenters, by determining the "water requirement" or ratio of the weight of water absorbed

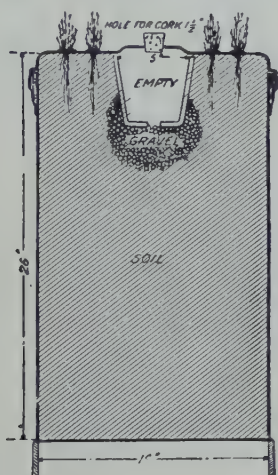


FIG. 26.—Briggs and Shantz experiments—arrangement.

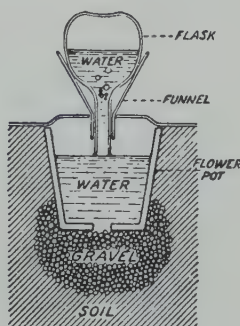


FIG. 27.—Briggs and Shantz experiments—device for adding water.

by a plant during its growth to the dry matter produced (or sometimes to the weight of grain).

An excellent summary of such data was made by Briggs and Shantz in 1913,¹ including the results of some extended observations made by them at Akron, Colo., in 1913, for most of the ordinary field crops and grains. They grew the plants in galvanized-iron cans 16 in. in diameter and 26 in. high filled with soil. All direct soil evaporation was practically eliminated by tight covers and by sealing the opening around the stems of the plants with wax. The water used during growth was accurately measured and supplied as needed by a simple device which eliminated any loss by evaporation. The cans were exposed in open sheds so as to give normal growing conditions without including any

¹ Bur. Plant Industry, "Water Requirement of Plants," U. S. Dept. Agr., *Bulls.* 284 and 285.

rainfall. Over 200 cans were used in these experiments, each plant or crop being grown in six cans to eliminate error in the results (see Figs. 26 and 27).

Certain of the results of these experiments are given in Table 34.

TABLE 34.—WATER REQUIREMENTS OF CROPS (*Briggs and Shantz, 1913*)

Crop	Water requirement	
	Based on weight of dry matter produced	Based on weight of grain produced
Alfalfa.....	1068	
Clover.....	709	
Rye.....	724	2215
Oats.....	614	1680
Wheat.....	507	1357
Corn.....	369	
Potatoes.....	448	
Weeds.....	322	

Where data of crop yields are available, transpiration losses can be estimated fairly well by such data as are given in the foregoing table. In Table 35 is an example of such an estimate applied to a hypothetical area of 100 sq. miles.

TABLE 35.—EXAMPLE OF COMPUTATION OF TRANSPIRATION LOSSES

Crop or condition of area	Area, square miles	Crop yield per acre	Water requirement	Transpiration losses	
				Inches depth on crop area	Inches depth on 100 sq. miles
Wheat.....	30	20 bu.	1,357	7.15	2.14
Rye.....	15	14 bu.	2,215	7.75	1.16
Clover or grass.....	20	1.0 ton	709	6.25	1.25
Pasture.....	15			5.0±	0.75
Woods and brush.....	20			9.0±	1.80
Total.....	100				7.10

In the above table wheat and rye are assumed at 60 and 56 lb. per bushel, respectively. Pasture losses are assumed at somewhat less than grass land. Woods and brush are estimated for the entire year. The present limitations of this method are obvious, but with more experimental data available it can be better applied. Furthermore, crop yield is a logical measure of transpiration losses.

STREAM FLOW OR RUN-OFF

As has already been noted, stream flow or run-off which may be of service in producing water power is made up of surface flow plus percolation, the latter two factors representing broadly the residual of precipitation minus water losses.

Surface flow is the portion of rainfall or melted snow which reaches the streams by flowing over the surface of the ground, including, of course, such rain as falls directly on the stream itself or its tributary ponds, lakes, or reservoirs. Surface flow commonly constitutes the greater portion of stream flow, although where geological and soil conditions favor rapid percolation and large underground storage exists, as, for illustration, in deep deposits of gravel or coarse sand, percolation may equal or even exceed surface flow.

Percolation is the ground-water flow to the stream, which is going on continuously. In periods of drought and especially in the growing season, when water losses are at a maximum and surface flow nearly or wholly ceases, percolation alone must supply the stream flow. Hence, the minimum flow of a given stream (where not regulated by storage) usually approximates a certain amount which is fixed by the character and extent of its ground-water storage but which will obviously vary greatly for streams with different geological and soil conditions.

Surface flow is the variable portion of stream flow and by its fluctuations causes the great variations commonly found in the latter. The various factors causing variation in surface flow have already been discussed in detail and are outlined on page 74.

Determination of Surface Flow and Percolation. *Analysis of Stream-flow Data.*—It is possible to approximate roughly from stream measurements the proportion of stream flow which is percolation by considering the latter to be indicated in amount by the low-water yield. For illustration, the measurements of the Souhegan River at Merrimac, N. H., show the following:

TABLE 36.—DISCHARGE OF SOUHEGAN RIVER AT MERRIMAC, N. H.
(Drainage area = 168 sq. miles)

Year, October to September inclusive (1)	Precipitation at Concord, inches (2)	Discharge, second-feet		Approximate ratio of perco- lation = col. 4 col. 3
		Mean (3)	Approximate minimum (4)	
1913-1914	36.18	318	35	0.11
1914-1915	34.56	240	35	0.15
1915-1916	43.26	382	35	0.09
1916-1917	30.48	260	35	0.13
1917-1918	31.57	216	35	0.16
Mean.....	35.21	283	35	0.12

It is probable that the average percolation for the year is somewhat greater than that during the dry season, when the ground-water level is tending toward a minimum. On the other hand, the minimum flow assumed in the above table is somewhat above the actual minimum in extreme low water, which was about 22 sec.-ft. and occurred in October, 1914. As will be noted, percolation comprises about 12 per cent of the total run-off of this stream. The student should make similar studies for other streams where data of flow are available, taking care to avoid cases where low-water flow is affected by artificial storage.

In a report of the Miami Conservancy District¹ are given similar data for streams in the Miami Valley, based upon hydrograph studies, from which Table 37 is abstracted:

TABLE 37.—SURFACE FLOW AND PERCOLATION—MIAMI VALLEY

River	Period	Drainage area, square miles	Precipitation at Dayton, inches	Surface flow		Percolation		Total run-off, inches
				Inches	Per cent of total	Inches	Per cent of total	
Miami at Dayton.....	1894-1919	2,525	37.1	7.77	65.6	4.08	34.4	11.85
Buck Creek at Springfield.....	1915-1919	163	37.8	4.83	44.3	6.06	55.7	10.89
Mad River at Wright.....	1915-1919	652	37.8	7.34	53.3	6.44	46.7	13.78
Stillwater River at West Milton.....	1915-1919	600	37.8	10.70	78.8	2.88	21.2	13.58

The wide differences in the proportions of surface flow and percolation in the above table are due to different geological and soil conditions. In the Mad River Valley there is relatively large underground storage in deep deposits of glacial gravel, while the comparatively loose and shallow surface soil permits rapid percolation. Gravel deposits are less extensive in the Miami Valley above Dayton and still less frequent in the Stillwater Valley. Over a considerable portion of the latter basin there are but a few feet of residual clay soil overlying the bed rock, which generally is limestone. On these drainage areas surface slope has but little influence on run-off. In fact the surface slopes are steeper over the Mad River drainage area, where percolation is great, than on the Stillwater where flood run-off predominates.

It is evident from the few examples given that the proportions of surface flow and percolation will vary through wide limits, percolation being small in the case of relatively impervious soils and subsoils, especially where slopes are steep and vegetation lacking but reaching an amount of 50 per cent or more of the total run-off where conditions especially favor it.

¹ "Rainfall and Run-off in the Miami Valley," *Tech. Reports*, Part VIII, pp. 164-167, 1921.

Measurements of percolation may be made directly in the field laboratory by the use of the lysimeter as previously noted, although, unless arrangements are made to remove excess surface water, the results are not necessarily comparable with actual percolation as occurring on land areas, especially with sloping surfaces.

Measurements of surface flow may also be made directly by collecting and measuring the surface flow from a small segregated area. Such experiments have been carried on by the Miami Conservancy District and are described in Part VIII of their *Technical Reports* previously mentioned.

In this case four plats, each 5 ft. square, were located in open places at Moraine Park near Dayton, two on level ground and two on a hillside, the two sets being about 100 ft. apart and the two plats of each set about 10 ft. apart, with a standard rain gage near each set. The slope of the ground on the hillside was about 18 per cent. One plat on the hillside and one on the level ground were located where the surface covering was sod; the other two were located where the sod had been removed leaving bare soil. The soil was mostly sand and gravel and favored considerable percolation. The plats were isolated from the adjacent ground by corrugated iron strips set into the ground about 8 in. and extending above it about 4 in., taking care in setting the strips not to disturb the ground inside the plats. Concrete was placed around the outside of the corners, to prevent leakage at the joints. A galvanized iron tank, 18 in. in diameter and 4 ft. deep (with tight-fitting cover), to catch the surface run-off, was set in the ground just outside the lower corner of each plat and was connected to the inside of the plats by a 3-in. vitrified pipe laid in concrete.

Certain of the conclusions reached from these Moraine Park experiments were as follows:

For small areas the occurrence and amount of run-off are affected much less by surface slope than by surface cover.

Appreciable surface run-off frequently occurs during intense summer storms, when the upper 6 in. of soil are not nearly saturated.

Surface run-off does not occur during some less intense storms, even though the ground is saturated.

Water can be absorbed by the bare soil at times when the soil is unusually dry at a rate as great as 1 in. per hour for intervals as long as 30 min.

Water cannot be absorbed by the bare soil at any time, no matter how dry it is, at a rate as great as 3 in. per hour for periods as long as 5 min.

The annual surface run-off amounts to about one-eighth of the rainfall, and the annual percolation amounts to about one-fifth of the rainfall.

Another series of experiments was made at this same time where artificial rainfall was obtained by means of sprinkling cans, using the same general method of measurements of surface run-off as previously

described. Some of the conclusions from these sprinkling experiments were as follows:

The rate of surface run-off increases as the rate of rainfall increases, the former being directly proportional to the latter when the surface soil is saturated. An approximate average relation of $x = y - 0.30$ was obtained, where x and y are, respectively, rates of surface flow and rainfall in inches per hour.

The rate of percolation when the surface soil is not saturated increases as the rate of rainfall increases, the variation being according to a straight-line equation and the rate of increase being proportionally greater for loose loamy soil than for heavy clay soil.

Cultivation has a relatively important effect in reducing the amount of surface run-off.

Much valuable information in detail is given in this report and, as suggested therein, more experimental data are needed along these lines under different conditions of surface and soil. By installing a lysimeter it is possible to check up the amount of percolation and compare it directly with the measured surface run-off and thus obtain information of much value in studies of the hydrology of any given area.

Effect of Forests on Stream Flow.—This phase of hydrology has excited much popular interest as well as controversy, opinions going to extremes, on the one side claiming that forests are great natural reservoirs which conserve and regulate the disposal of the rainfall, to appear later as stream flow, while others consider that forests not only decrease stream flow but increase flood tendencies as well.

It is very commonly believed that small streams are much diminished in flow and frequently dry up and disappear when a region is deforested, this opinion usually being based upon observations of some old inhabitant made years apart with large reliance on the definite remembrance of past conditions. It is needless to say that such proofs are unreliable and the true effect of deforestation can be determined only by careful studies of water losses and run-off and this effect is often slight in amount and difficult to perceive. Unfortunately too, advocates of forestry in their zeal for forest preservation often make broad and unqualified statements characterizing all forests as water conservators and using this often erroneous statement as their chief argument to justify continued forestation, instead of allowing that worthy objective to stand, as it should, on its own merits.

Agreement is general that changes in forest conditions in this country have had no appreciable effect on rainfall, certainly as far as records show, although long-time information of value in such a study is meager. In the tropics, however, the presence of dense forests over large areas may actually affect the amount of rainfall, but here again this is mere conjecture, although reasonable.

The effect of forests on stream flow is due not only to the growing trees and vegetation but also to the so-called "forest floor" or layer of

leaves, litter, roots, organic material, etc., on the surface of the ground. This forest floor tends to retard surface flow as well as percolation by so-called "ground storage," but to a greater or less degree varying with the character of the subsoil, whether clayey and impervious or sandy and porous. In very heavy rainfalls, too, the forest floor becomes so saturated that surface flow is but little retarded. Thus ordinary floods are lessened in severity by forests while very great floods are but little mitigated in effect. Snow storage, however, is usually more effective and continued in forests, tending to a decrease in flood tendency in the spring months upon forested areas in northerly latitudes. This tendency is partially offset, however, in the West, in the mountains, where great drifts of snow form in the open which, on account of their great depth, are slow in melting and thus tend toward a more gradual run-off. On the whole, forests tend to reduce flood flow.

The effect of forests upon the average and low-water yield of streams can be best understood by a consideration of relative water losses on forested and open areas. Soil evaporation will be less in the forest due to low temperatures and greater relative humidity. This was clearly demonstrated in Ebermayer's experiments referred to on page 90 and confirmed by other less complete observations in this country. Interception and transpiration losses are larger on forested areas and in fact only exceeded by those of certain cropped areas or of grass and grains. The net result may, however, show a lesser total water loss for forested areas and, therefore, a somewhat greater average stream flow than for the typical open-country area, especially where the latter contains much pasture and grass land and grain crops.

Thus Ebermayer's lysimeter experiments for a 4-ft. depth of soil (which, however, eliminated surface-flow comparisons) showed the following water losses:

In the open..... 16.3 in. (evaporation, transpiration, and interception).
 In the forest..... 17.8 in. (interception = 7.5 in., other losses = 10.3 in.).
 (His stations in the open were mostly in meadow land.)

Fernow¹ gives the following ratios of evaporation (*i.e.*, retention or total water losses) compared with that of water area:

Water.....	1.0
Bare soil.....	0.6
Sod.....	1.92
Cereals.....	1.73
Forest.....	1.53

Horton² found, however, in a study of the run-off of Michigan streams that the effect of deforestation was less than that of subsequent drainage and tillage, and the run-off exceeded that when forested conditions pre-

¹ U. S. Dept. Agr., U. S. Forest Service, *Bull.* 7.

² *Proc. Mich. Eng. Soc.*, 1908, pp. 176-195.

vailed, due chiefly to lessened interception and transpiration losses. For an assumed yearly precipitation of 36 in. he found run-off in inches to be:

Area	Conditions		
	Forested	Bare	Present
Hardwood.....	10	19	15
Pine.....	13	20	15

It is evident that there is no great difference in water losses on forested and open areas, all things considered, and the advantage in respect to average run-off is not always with the forest, although the flood flow of the latter is nearly always less and in general the stream flow less fluctuating. Forests are important, however, in preventing or checking soil erosion and lessening the amount of silt borne by streams, which is often of importance in power as well as navigation projects.

A vigorous and comprehensive discussion of the subject, entitled "Forests, Reservoirs, and Stream Flow," will be found in *Transactions* of the American Society of Civil Engineers (vol. 42, p. 245 *et seq.*, 1909). More definite and complete data over longer periods of time are needed to better establish the net effect of forests on stream flow. It must also be kept in mind that the percentage change in the forest and other conditions of large drainage areas—like the Connecticut or Merrimac in New England or the Hudson in New York—has been but little during even the last hundred years, while few data of precipitation go back that far and no run-off records exceed some 50 years of time.

Run-off Formulas.—Many attempts have been made to express in equational form stream flow as a function of precipitation, bringing into such equations one or more of the meteorological or other factors which affect run-off.

Vermeule¹ made a careful study of the precipitation and run-off of streams in the Northeast, as a result of which he proposed the following as representing annual evaporation (or water losses) in inches:

$$E = (11 + 0.29R)M \quad (11)$$

in which R is precipitation in inches and M is a factor depending on mean air temperature as follows:

Temperature, Degrees Fahrenheit	Factor M
40	0.77
45	0.91
50	1.07
55	1.26
60	1.47

¹ *Geol. Survey N. J.*, vol. 3, 1894; also 1899.

For certain streams (the Sudbury, Croton, and Passaic rivers) he deduced the formula $E = 15.50 + 0.16R$ for annual water losses and also a similar formula with variable coefficients for each calendar month. These formulas give fairly consistent results, at least in estimating yearly run-off, on the streams which supplied the basic data of flow and streams of similar regimen. They are likely to result in large error when used generally, even for yearly values, and are practically useless for estimating monthly run-off, depending, as it does, both upon the precipitation of previous months and so many other factors the effect of which is not included in the formula.

Justin¹ suggested the following, based upon precipitation and run-off data for 14 streams in the eastern United States:

$$C = 0.934S^{0.155}\frac{R^2}{T} \quad (12)$$

where C and R are, respectively, annual run-off and rainfall in inches, T is mean annual temperature in degrees Fahrenheit, and S is the mean slope of the drainage area, assumed to be the maximum difference in elevation divided by the square root of the area. He did not attempt to formulate monthly run-off but considered his formula good within 10 per cent for yearly values. It will be found, however, even on streams considered by him, to depart more than this from observed values for single years, although the mean of several years shows good agreement.

As will be noted, Justin's formula is in the shape of $C = K R^2$ for a given drainage area (S being a constant and T nearly so) where K reflects the characteristic peculiarities of the area. Using a mean value of K , as he does, obviously involves approximation when applied to a particular stream.

Vermeule's formula correctly makes run-off a residual, *i.e.*, of precipitation and water losses but utilizes only one of the factors affecting water losses (although perhaps the most important), *viz.*, temperature. Justin's formula is not in correct form, in that run-off is made a proportion of precipitation, although the effect of slope as well as temperature is included. Neither formula takes into account the many other factors besides temperature which affect water losses nor are ground-water conditions considered, all of which are of great effect in modifying run-off. Furthermore, the necessary limitation of such formulas to yearly run-off of itself prevents their practical use in water power problems, many of which require in their solution monthly, weekly, and even daily data of flow.

Sherman² has suggested the use of a "unit graph" representing 1 in. of run-off, based upon an observed hydrograph of run-off, for a 24-hr.

¹ *Trans. A.S.C.E.*, vol. 77, p. 346 *et seq.*, 1914.

² SHERMAN, L. K., "Stream Flow from Rainfall by Unit-graph Method," *Eng. News-Rec.*, Apr. 7, 1932, and "The Relation of Hydrographs of Run-off to Size and Character of Drainage Basin," *Trans. Amer. Geophys. Union*, 1932, pp. 332-339.

rainfall. The unit graph together with the observed daily rainfalls is used in estimating daily run-off. For a given drainage area the unit graph remains the same but varies in form for different rivers, depending upon steepness of slopes, pondage, etc. This method appears to be of value in run-off studies.

Computing Run-off from Precipitation and Water Losses.—Meyer¹ uses what may be termed a hydrophysical method (also previously employed in principle by Rafter and Horton and various English engineers), computing water losses and deducting them from precipitation to determine run-off, also taking into account ground storage and percolation. His evaporation- and transpiration-loss curves are based on both theory and experimental results, modified and revised to accord with various actual stream-flow data.

Transpiration as used by Meyer also includes interception losses, although the investigations made by Horton (see pages 92–98) indicate that these losses follow quite different laws.

Meyer's method is logical but lacks for its application accurate values of many of the factors used. His mean curve of soil evaporation, for illustration, is to be applied by means of coefficients varying from 0.95 to 1.25, to be selected practically by judgment. His mean transpiration curve is described on page 98. At present his methods are of little value in independently estimating stream flows, except for very rough results, but may be of value in checking and analyzing stream-flow records and helping to account for differences in such records, as well as logically applying them on other streams, keeping in mind, however, that this can, at present anyway, only be for yearly run-off or possibly the two or three main subdivisions of the "water year" to be subsequently explained.

Run-off from Melting Snow.—A study of this subject² indicates that the approximate peak run-off under snow-covered conditions without rain can be predicted a day or two in advance by using the value of 0.020–0.025 in. per degree-day excess above 27° based upon the mean 24-hr. temperature. (Note that mean daily temperature is closely the mean of the observed maximum and minimum.)

For smaller drainage areas at high elevations the degree-day relation appears to run higher than 0.025.

For moderate amounts of rainfall, say, up to 1 in., the run-off degree-day relation also holds for peak-day run-off if half the precipitation is deducted in computing run-off.

More study and information are desirable upon this subject.

Determination of Drainage-area Characteristics.—There are certain physical characteristics of drainage areas necessary to determine in studies of water losses and run-off. These are: (1) average elevation or

¹ *Trans. A.S.C.E.*, vol. 79, p. 1056 *et seq.*, 1915; also MEYER, "Hydrology."

² *Jour. Boston Soc. Civil Eng.*, September, 1930, pp. 354–362.

altitude, (2) mean land slope; (3) mean stream slope, (4) drainage density, and (5) drainage-area characteristics.

A good topographic map—those of the U. S. Geological Survey are excellent—is the basis for much of the above information. Some of the methods applicable are as follows:

1. *Average Elevation or Altitude*.—For small areas this may be determined by measuring with opisometer the length of all contours on flat areas or the length of say 100-ft. contours on steeper areas. The weighted mean elevation is the sum of the contour lengths $\times$ respective elevations $\div$ total length of contours. For large areas the area may be divided into squares of 1- to 10-mile sides, depending on the scale, the elevations at the intersections of all squares tabulated, and then averaged.

2. *Mean Land Slope*.—For small areas the average distance between contours (horizontally) may first be found by measuring, with opisometer, contour lengths (or say 100-ft. contours on steep areas). Then,

$$\text{Slope} = \frac{\text{contour interval}}{\text{average distance between contours}} \quad (13)$$

where

Average distance between contours =

$$\frac{\text{area}}{\text{total length of contours of given interval}}$$

For large areas, subdivide into squares and count the number of contour intersections. Then (for one square)

$$\begin{aligned} D_A &= \text{average distance between intersections} \\ &= \frac{\text{total length of sides of square}}{\text{number of intersections}} \end{aligned}$$

And the mean distance between contours

$$= D_A \sin 45 \text{ deg. (approximately),}$$

and as before,

$$\text{Slope} = \frac{\text{contour interval}}{\text{average distance between contours}}$$

For very large areas use sample areas, properly chosen and averaged, determining their elevation by the contour measurement method. For the ordinary 1:62,500 map sample squares of from 1 to 3 in. in size for, say, each 100 sq. miles of area will usually give good results.

3. *Mean Stream Slope*.—This may be obtained by tabulating lengths and elevations of the stream and its principal tributaries and determining a properly weighted mean slope.

4. *Drainage Density* (or Length of Stream Channels per Unit of Area).—This may be determined by dividing the total length of streams,

determined by opismoter or scaling, by the area of the basin in square miles.

5. *Drainage-area Characteristics.*—The general nature of the drainage area—the proportions wooded or in cultivation or grass land, or occupied by cities and towns, etc.—may be determined by field reconnaissance and the use of the topographic map. Its condition of culture, kind, and extent of crops, etc., may also thus be determined. The soil maps and crop reports of the U. S. and state departments of Agriculture and the geological map of the U. S. Geological Survey will be found useful, where available.

Example of Water Losses—Run-off Study.—In Table 38 are given some of the details of a study of water losses and run-off for the Seneca and Owasco rivers, two nearly adjacent streams in central New York. These were made by the author in collaboration with Robert E. Horton in 1915, during the estimation and valuation of the water power on Seneca River at Seneca Falls taken by the state of New York, due to the construction of the State Barge Canal.

A 5-year record of flow was available for Seneca River and a 2-year record for the Owasco. Yearly precipitation on the Owasco River was nearly 3.5 in. greater than on the Seneca basin and the run-off nearly 7 in. greater. The study of water losses was made to ascertain the reasons for the relatively greater run-off of the Owasco basin over that due to somewhat greater precipitation, which was found to result essentially from the conditions given in Table 38.

1. A greater percentage of cultivated land on the Seneca basin and more grass land on the Owasco, resulting in greater water losses for the former basin.

2. A deep till loam on most of the Seneca basin, favoring soil evaporation, and Volusia silt loam, closely underlain by shale rock on the Owasco, minimizing soil evaporation and favoring quick run-off.

To determine basin conditions reconnaissance was made by roads and railroads; soil maps and data of crop yield were used in estimating water losses and geological maps in studying soil conditions.

The results were of value in explaining the differences in measured run-off of these two basins. As will be noted, only yearly averages were obtained, which were, however, of use in this case in estimating available water power because of the high extent of regulation of Seneca River, due to the large storage afforded by Seneca Lake.

Necessity for Data of Actual Stream Flow.—It is obvious from the foregoing discussion that no accurate method is available for the indirect estimation of stream flow based upon the consideration of precipitation and either run-off factors or water losses. While fair approximations may be made in this way of annual run-off, more detailed estimates by months or weeks (which are usually required in water power studies) are out of

TABLE 38.—STUDY OF YEARLY PRECIPITATION, RUN-OFF, AND WATER LOSSES OF SENECA AND OWASCO RIVERS, NEW YORK

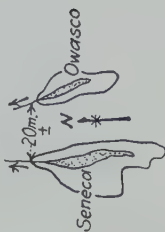
Condition of drainage area	Per cent of land area	Losses, inches						Precipitation, inches	Computed land run-off, inches	Measured run-off, inches				
		Soil evaporation		Interception		Transpiration				Total losses, inches	(8)	(9)	From land and water area (10)	From land area (esti- mated) (11)
		Total	Weighted	Total	Weighted	Total	Weighted							
(1)	(2)													
Seneca River: Water and swamp area = 85 sq. miles (11 per cent); land area = 693 sq. miles (89 per cent)														
Woods.....	15	4	0.6	8	1.2	9	1.4	21	3.15					
Grass.....	40	6	2.4	2.5	1.0	3.5	1.4	12	4.80					
Cultivated.....	43	11	4.7	5	2.1	8	3.4	24	10.30					
Villages, etc.....	2	10	0.2	1	0.0	2	0.1	13	0.26					
Totals.....	100	..	7.9	...	4.3	...	6.3	..	18.51	34.57	16.06	(1910-1914) 13.5	15.0	
Owasco River: Water and swamp area = 11 sq. miles (5 per cent); land area = 195 sq. miles (95 per cent)														
Woods.....	20	4	0.8	8	1.6	9	1.8	21	4.20					
Grass.....	60	6	3.6	2.5	1.5	3.5	2.1	12	7.20					
Cultivated.....	18	9	1.6	5	0.9	8.0	1.4	22	3.96					
Villages, etc.....	2	10	0.2	1	0.0	2.0	0.1	13	0.26					
Totals.....	100	..	6.2	...	4.0	...	5.4	..	15.61	37.90	22.29	(1913-1914) 20.4	21.3	

NOTE: Average slope of drainage areas: Seneca = 6.25 per cent; Owasco = 6.6 per cent.

Drainage density (stream length per square mile): Seneca = 1.48 miles; Owasco = 1.85 miles.

Soil conditions: Seneca mostly deep till loam, especially adapted for cultivation. Owasco mostly Volusia silt loam, not well adapted for cultivation and with underlying shale rock near surface.

Evaporation from water surface assumed at 33 in. yearly (as at Rochester).



Approximate sketch of drainage areas

the question, at least with our present limited knowledge concerning water losses and drainage-area characteristics. In fact, the relations of run-off to precipitation are so complex and variable that even adjacent drainage areas will frequently show quite different results. As may be well stated, every stream is a law unto itself, the peculiarities of which can be accurately determined only by continuous measurements and records of its daily flow extending over a long period of time. The detailed past performance of the stream as thus determined by measurement is the best index or measure of what may be expected in the future.

U. S. Geological Survey Water Supply Papers.—The importance of procuring systematic records of stream flow as a basis for the proper design of works for water supply, irrigation, and water power, as well as for use in all general problems involving the water resources of this country, led to the development of this work by the U. S. Geological Survey about 30 years ago, since which time their annual reports or *Water Supply Papers* have provided data of stream flow at stations on many of the more important streams of the country.

The majority of these river stations are "current meter stations," *i.e.*, a discharge-rating curve is developed and corrected if necessary from time to time by measurements of discharge with current meter by some engineer of the Survey. Daily gage heights (often an automatic gage record) are obtained by a local observer, which with the discharge-rating curve furnish a fairly accurate basis for estimating the daily flow of the river. A portion of the river stations are at dams or hydroelectric plants where the Survey cooperates with the owner in procuring and publishing records of flow. To extend the scope of this work the Geological Survey also cooperates extensively with the various states and, in addition to data of yearly flow, has published many special reports in which precipitation and stream-flow records are compiled over periods of time and attention is also given to the water power and storage resources of selected areas or states.

At present the annual reports are issued on the basis of 12 major drainage areas, with a report for each area as follows:

- I. North Atlantic.
- II. South Atlantic and Eastern Gulf of Mexico.
- III. Ohio River.
- IV. St. Lawrence River.
- V. Upper Mississippi River and Hudson Bay.
- VI. Missouri River.
- VII. Lower Mississippi River.
- VIII. Western Gulf of Mexico.
- IX. Colorado River.
- X. Great Basin.
- XI. California (Pacific slope).
- XII. North Pacific coast.

The earlier reports are now only available at libraries; those for later years may be obtained from the Director of the U. S. Geological Survey, Washington, D. C., or, where the supply has become limited, at a nominal price from the Superintendent of Documents, Washington, D. C.

The student should familiarize himself with these *Water Supply Papers*, preferably by procuring some recent issue for a portion of the country with which he is acquainted and studying the amount, arrangement, and kind of data thus made available and the methods used in both the field and office. In nearly all cases these *Water Supply Papers* are the only source of information on stream flow that is available in this country, although occasionally such information is obtained and published in reports by state or municipal authorities (as, for illustration, the Metropolitan District Commission of Boston, which annually publishes the precipitation and run-off of the Sudbury, Nashua, and other rivers).

In Canada systematic measurements of stream flow (as well as precipitation) and annual publication of data in similar form to that of the U. S. Geological Survey are under the direction of the Dominion Water Power Branch (Ottawa), who now have information available for several years.

The Water-year.—In compiling and comparing precipitation and run-off records it will be found, as previously noted, that the water losses and ratio of run-off to rainfall will vary greatly during the different seasons of the year, although for a given season, like winter, this variation is not nearly so great. Rafter and Vermeule, therefore, suggested the use of a "water-year," rather than a calendar year, as more logical in studies of hydrology.

As used by Rafter the water-year is commonly taken as beginning with Dec. 1, although in some cases in the more northerly states it may be desirable to begin the year with Nov. 1. The year is divided into three periods called the storage, growing, and replenishing periods.

The storage period includes the months from December to May, inclusive, during which evaporation and vegetation demands for water are relatively slight and a very large proportion of the precipitation appears in the streams as run-off. In cases where the discharge of a river is controlled by storage reservoirs, they are usually filling during this period, often largely from the results of snow storage.

The growing period, from June to August, inclusive, includes the period when vegetation is growing and all water losses are at a maximum. During this period often not more than 0.1 of the rainfall appears in the streams, and sometimes this proportion may be as low as 0.05 or even less. During the growing period the ground-water level tends gradually to lower unless the rainfall is greater than the average.

In the replenishing period, September to November, inclusive, with normal rainfall the ground-water level tends to rise again, and the run-off is larger than in the preceding period. The term "replenishing" is used because there is a tendency during this period to return to normal conditions.

The time of beginning and ending of these three periods will vary in different portions of the country and may be determined for a given locality from a knowledge of the precipitation and temperature conditions.

This method of dividing the year is of especial advantage where, for lack of sufficient data of run-off, estimates of discharge have to be based primarily upon precipitation data. It is usually found that the ratio of run-off to precipitation for one of these periods is fairly constant, whereas, as previously noted, there is a great difference in this ratio for the different periods of the year, as well as for individual months during a given period.

Another advantage of arranging run-off and precipitation data according to the water-year is that little or no discrepancy occurs (as is the case when such data are arranged by the calendar year) on account of precipitation in the form of snow appearing as run-off during the next year.

It will often also be found convenient and sufficiently accurate to combine the months of the growing and replenishing periods and divide the water-year into two 6-months periods—a winter and summer, so called—instead of the three periods.

The U. S. Geological Survey in its *Water Supply Papers* is now using a water or climatic year rather than a calendar year for its annual reports, beginning with Oct. 1, so that the 1918 report, for illustration, actually contains stream-flow data from October, 1917, to September, 1918, inclusive.

Essentials of Stream Flow for Water Power Studies.—This subject will be considered in more detail later, but, in order properly to relate the study of hydrology to the design of water power plants, it is desirable to outline the essentials of stream-flow data required in such work. They are as follows:

1. Daily (weekly or monthly) flow over a period of years, the longer the better, as a basis for plant capacity and estimated output, both of which are a function of the average flow of the stream as well as its distribution during the year, as shown by flow-duration curves. If a storage project is under consideration, the records should include one or more dry periods of years.

2. Minimum or low-water flow as a basis for the amount of primary or dependable power and (often) size of steam auxiliary.

3. Maximum or flood flow, to enable a safe installation with adequate spillway or gate relief and avoidance of damage by flowage to property upstream.

Those three essentials may all be afforded by a long record of stream flow, but if this is not long enough to give closely the extremes of flow and high water to be expected, these must be estimated in some indirect manner. This is most frequently the case with flood flows for which the extremes may occur rarely and yet must be provided for.

Minimum or Low-water Flow.—The minimum flow of a stream without storage is, as already stated, that due exclusively to ground-water yield or percolation. For different streams this amount may vary greatly due especially to differences in factors affecting the rate and amount of percolation. For a given stream, however, the minimum flow approaches a fairly constant amount, as Vermeule showed by plotting run-off for a summer period of several successive months against time abscissas (a hydrograph). He found that normally after 2 or 3 months of ordinary dry weather the resulting curve would be substantially horizontal. The low-water yield of a stream without storage may, therefore, be determined quite closely by measurements after a period of drouth has prevailed for some time.

Where the low-water yield of a stream has not actually been determined by measurements, it may be approximated from the low-water yield of other streams of similar characteristics, although this may be difficult for lack of proper comparative data and the results, therefore, uncertain. In any water power project where the amount of primary power must be known with reasonable accuracy, reliance should be made only on actual measurements of the low-water flow.

Minimum flow may vary from zero in the case of streams of very small drainage area or even of large streams in the West, to from 0.3 to 0.5 sec.-ft. per square mile of drainage area or more, where conditions of precipitation, subsoil, etc., may favor a well-sustained low-water flow.

With artificial storage the minimum flow of a stream can of course be increased to any practicable amount, limited only by the average yield of the stream and the distribution of this yield throughout the year, and the cost of storage works.

Maximum or Flood Flow.—The maximum flow of a stream is affected by many factors. Primarily it will be occasioned by excessive rainfall or perhaps a combination of melting snow and large rainfall, but the shape of the drainage area and steepness of slopes, the presence of artificial storage, the path of the storm, and other conditions affecting run off, as previously described, will have much effect upon flood tendency.

At times of extreme flood, evaporation and percolation effects are reduced to a minimum and the proportion of rainfall reaching the streams may be very much greater than the normal. This is especially true in the case of an unusual flood occurring during the winter time in the northern states, when the ground is frozen and nearly all of the precipi-

tation gets to the streams soon after falling. On the other hand, many great floods have occurred during the summer time or fall and have been the result of very excessive rainfall occurring during a wet period, when the soil was already well saturated. The subject of maximum rainfall has already been discussed on pages 62-71.

The most satisfactory allowance for flood at a given power site on a river would evidently be that based upon the maximum observed flood at that point or vicinity, provided that the time covered by such observations or records was adequate. In many cases, however, such information is either entirely lacking or unreliable, and it becomes necessary to utilize

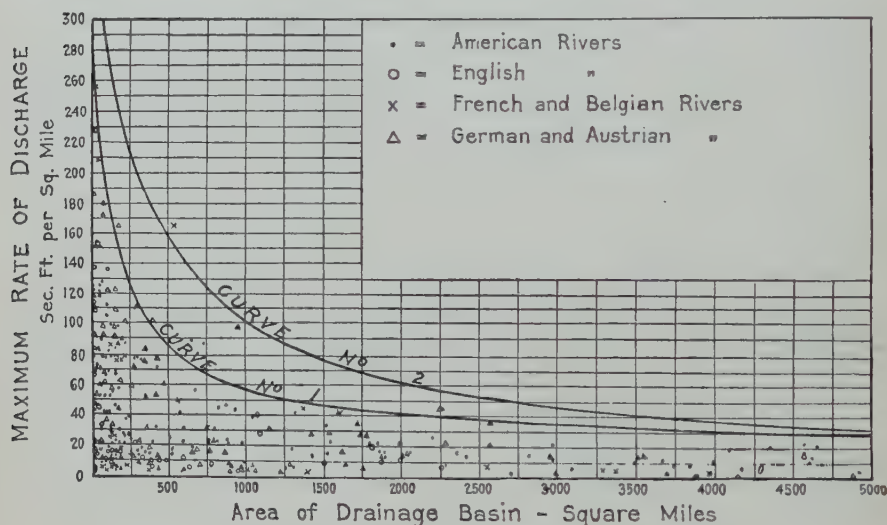


FIG. 28.—Rates of maximum flood discharge. (Kuichling.)

information at other points on the river or other streams, or perhaps even more indirect methods.

Methods of Estimating Flood Flow. *Formulas Involving Drainage Area Only.*—As has already been noted, excessive rates of rainfall usually apply only to limited areas, and a storm extending over a large river basin would have an average rate of precipitation much less than the maximum. From this fact, too, the liability of small drainage areas to very high maximum rate of run-off is evident. There is, therefore, a general relation between the size of drainage area and the maximum rate of run-off to be expected. This is shown on the accompanying diagram (Fig. 28), upon which are plotted the results of observations of maximum flow at a great many different stations for rivers of different size, prepared by Kuichling.¹

¹ See *Report of New York State Engineer on Barge Canal*, p. 844, 1901.

These curves expressed by formula are:

Curve 1.

$$Q = \frac{44,000}{M + 170} + 20 \text{ (occasional floods)} \quad (14)$$

Curve 2.

$$Q = \frac{127,000}{M + 370} + 7.4 \text{ (rare floods)} \quad (15)$$

where M is the drainage area in square miles.

It is to be noted that this diagram was intended to apply particularly to conditions similar to those in the Mohawk River Valley, New York.

Murphy, in *Water Supply Paper* 147, has made a similar investigation regarding streams in the northeastern United States and obtained a curve whose equation is

$$Q = \frac{46,790}{M + 320} + 15 \quad (16)$$

Obviously, such formulas and curves which merely take into account size of drainage area are safely applicable only to conditions similar to those upon the rivers from whose data of run-off they were derived and should be used with caution in estimating the probable maximum flow of a river of different regimen of flow.

Jarvis¹ has recently made a study of available flood data and concluded that the formula

$$Q = C\sqrt{M} \quad (17)$$

may be used to cover the entire range of floods to be expected by varying C between 10,000 as a maximum and 100 as a minimum. Such a formula takes into account size of drainage area to some extent but relies chiefly on assuming a proper value for C , which varies between very wide limits and, hence, is quite difficult to select properly. Jarvis has compiled and plotted all available flood data, which are shown on Fig. 29. As will be noted, this diagram contains many more data than that of Kuichling (Fig. 28)—that made available during the past 30 years.

Fuller's Flood Method.—Weston E. Fuller,² as a result of an elaborate study of flood data in the United States available through 1913, suggested a valuable method for the study of floods in which, for the first time, the factor of time of occurrence or probability was included. He divided conditions affecting floods into two general classes:

¹ JARVIS, C. S., "Flood Flow Characteristics," *Proc. A.S.C.E.*, December, 1924, p. 1545 *et seq.*

² *Trans. A.S.C.E.*, December, 1914, p. 564 *et seq.*

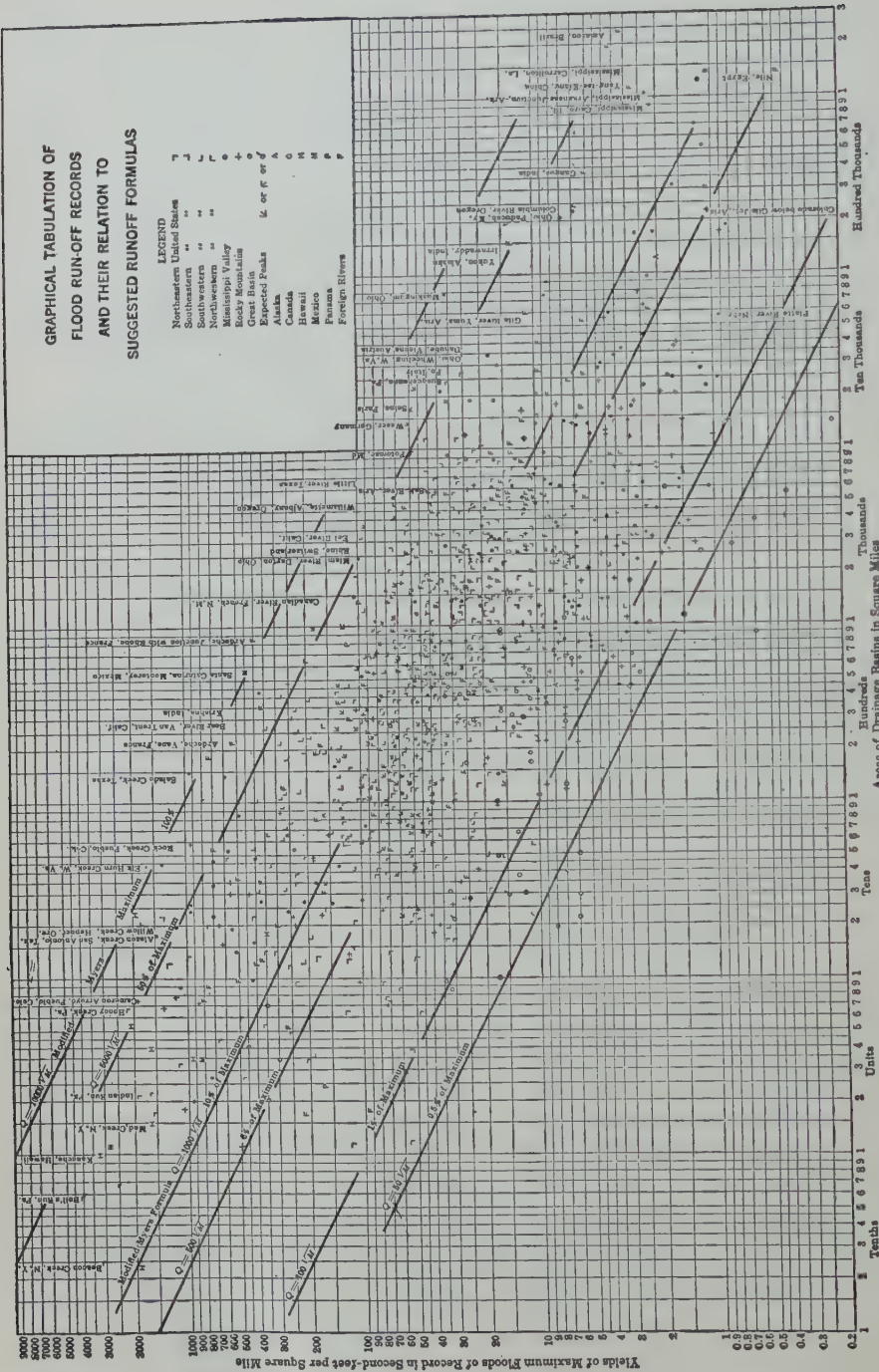


Fig. 20.—Flood flows and formulas. (Jarvis.)

1. Those relating to one stream tending to make all floods on it greater or less than on others. These are:

- Prevailing conditions of rainfall.
- Size, slope, and shape of drainage area.
- Character of soil and vegetation.
- Physical characteristics of channel.
- Reservoir capacity.
- Etc.

2. Those general in effect on different areas but variable in time. These are:

- Rate of rainfall.
- Snow conditions.
- Temperature conditions.
- Water stored in reservoirs, lakes, and ground.
- Velocity and direction of storm.
- Ice jams or temporary channel obstructions.
- Etc.

The effect of the first class of conditions with reference to a given drainage area is shown by the value of C in his formula, while the second class of conditions occurs sooner or later on all drainage areas and is, therefore, properly a function of time or T in his formula, or, to put it another way, no two floods are exactly alike and the number of combinations of circumstances affecting floods is infinite. If the conditions tending to large floods occur coincidentally with great rainfall, extraordinary floods are produced. This chance of coincidence is equal for different streams, its study one of probabilities.

The average yearly flood

$$Q_{ave} = CA^{0.8} \quad (18)$$

where C is a coefficient and A the drainage area in square miles, is the basis for his formula, obtained by logarithmic plots of actual flood flows, substantially all of which showed consistently common slopes ($0.8 \pm$) but different values of C . He concluded that 5 or 6 years of stream-flow records would fix Q_{ave} fairly well.

To obtain the greatest daily or 24-hr. flood in T years, the factor $(1 + 0.8 \log T)$ is applied to Q_{ave} . This relation was also obtained from logarithmic plots of actual flood ratios to Q_{ave} over different periods of time.

To obtain the crest flood or maximum during a portion of the day, the factor $(1 + 2 A^{-0.3})$ is further applied, also based upon such actual relations as were available for this purpose. This factor is perhaps the most uncertain of any that Fuller developed, owing to rather meager data.

The complete formula is then

$$Q_{\max} = CA^{0.8}(1 + 0.8 \log T)(1 + 2A^{-0.3}) \quad (19)$$

To show the effect of T and size of drainage area, Tables 39 and 40 are given.

TABLE 39.—RELATION BETWEEN FLOOD TO BE EXPECTED IN A SERIES OF YEARS AND THE AVERAGE YEARLY FLOOD
($Q = Q_{\text{ave}}(1 + 0.8 \log T)$)

Time, years (1)	Ratio of largest flood to average yearly flood (2)	Time, years (1)	Ratio of largest flood to average yearly flood (2)
1	1.00	50	2.36
5	1.56	100	2.60
10	1.80	500	3.16
25	2.12	1000	3.40

TABLE 40.—RELATION BETWEEN MAXIMUM FLOOD AND AVERAGE 24-HR. FLOOD
($Q_{\max} = Q(1 + 2A^{-0.3})$)

Catchment area, square miles (1)	Ratio of maximum flood to average 24-hr. flood (2)	Catchment area, square miles (1)	Ratio of maximum flood to average 24-hr. flood (2)
0.1	5.0	500	1.31
1.0	3.0	1,000	1.25
5.0	2.23	5,000	1.15
10	2.0	10,000	1.12
50	1.62	50,000	1.08
100	1.5	100,000	1.06

The values of C for different sections of the country are given in Table 41.

The use of the average yearly flood in his formula as a basic element is logical and gives for the drainage area in question a direct means of determining its particular flood tendencies. As many stream-flow records of 15 years extent and more are available, this portion of the formula may now usually be well determined.

The time or probability factor $(1 + 0.8 \log T)$, increasing as it does with T and at a rate of 0.8 for each multiple of 10 increase in time, would indicate that it is only a question of time when any given flood amount will be exceeded. This is not fundamentally correct, as every drainage area has some limiting flood possibility imposed by its location and character.

TABLE 41.—VALUES OF *C* IN FULLER'S FLOOD FORMULA

Locality	Number of stream stations considered	Value of <i>C</i>		
		Maximum	Minimum	Average
New England.....	32	110	17	57
Hudson River basin.....	14	132	37	85
Middle Atlantic states.....	31	140	30	76
South Atlantic states.....	49	113	25	66
Ohio River basin.....	38	142	47	81
St. Lawrence River basin.....	23	110	9.2	31
Hudson Bay basin.....	9	57	1.3	20
Upper Mississippi basin.....	16	55	7.4	22
Missouri basin.....	66	49	1.8	11
Lower Mississippi basin.....	8	52	1.7	17
Western Gulf of Mexico.....	4	31	9.5	17
Colorado River basin.....	24	46	3.0	18
Great Basin.....	45	55	0.6	13
Southern Pacific coast.....	29	193	15.8	68
Northern Pacific coast.....	52	186	2.2	45
Total or average.....	440	98	14	42

The relation of crest flood to maximum 24 hr. as given by $(1 + 2A^{-0.3})$ is also rough and may be altered by better and more complete data.

In spite of these defects Fuller's method of estimating flood flow is of value for approximate estimates and will become more so as more data become available to enable revision of his formulas.

The significance of *T* in his formula is that of a factor of safety or chance that can be taken in allowing for floods. If great damage or loss of life, or perhaps the wrecking or serious damage to power house or equipment, would be involved, use a large value of *T*. The added cost of using a large value of *T* is insurance against loss.

Boston Society of Civil Engineers Methods.—The Committee on Floods of the Boston Society of Civil Engineers made a study of flood run-off in their report upon the New England flood of November, 1927.¹

Assuming that the flood hydrograph is a triangle they find:

$$q = \frac{1290R}{T} \quad (20)$$

where *q* = the peak or crest flow in second-feet per square mile.

R = the flood run-off in inches on the drainage area.

T = the total flood period in hours (or the base of the flood hydrograph).

¹ *Jour. Boston Soc. Civil Eng.*, September, 1930, and December, 1932.

This formula was found to check well with the peak flows observed on numerous rivers.¹ It was also found that T could usually be well established by a simple flood hydrograph and tends to remain approximately constant, irrespective of the size of flood.

Where no flood hydrographs are available the committee suggested formulas derived from formula (20), as follows:

$$q = \frac{C_F R}{\sqrt{A}} \quad (21)$$

or

$$Q = C_F \sqrt{A} R \quad (22)$$

where (in addition to previous notation)

C_F = the "flood characteristic" or coefficient of a stream—ordinarily varying between 100 and 500 but with values in mountainous regions of 500 to 1000 and occasionally, with very flat streams, less than 100.²

A = drainage area in square miles.

Q = peak flow in cubic feet per second.

For New England values of R are as follows:

Occasional floods.....	$R = 3$ in.
Rare floods.....	$R = 6$ in.
Maximum flood.....	R not over 8 in.

*Major Pettis*³ developed the formula

$$Q = 328PW^{3/4}$$

in which W is the average width of the drainage area in miles, P is the storm rainfall in inches, and Q the maximum discharge in second feet.

He limits the use of this formula to drainage areas between 1000 and 10,000 sq. miles.

*Flood-characteristic Curves.*⁴—If both the time and quantity of flow in a flood hydrograph are divided by the square root of the drainage area, a new hydrograph will be obtained for a similar flood upon a similar drainage area of 1 sq. mile. If the quantities of flow in the new hydrograph for 1 sq. mile are again divided by the number of inches of flood run-off resulting from the flood, a hydrograph for a 1-in. flood on 1 sq. mile will be obtained. This is called the "Characteristic Flood Curve" and by this method different streams may be compared as to their drainage-area characteristics. Note that the peak point of this curve is the coefficient C_F in formula (22).

¹ *Ibid.*, p. 381, Table 19.

² *Ibid.*, p. 391, Table 22.

³ "A New Theory of River Flow."

⁴ *Jour. Boston Soc. Civil Eng.*, September, 1930, p. 392.

Rational Method.—The so-called “rational method” used in the design of storm-water drains for relatively small areas may also be applied in principle to the determination of maximum flow of larger streams, considering (1) the length of time required for water to reach the point in question from various subdivisions of the drainage area; (2) the amount of rainfall (usually by hours) and its duration; (3) the percentage flowing off; (4) the combination of the three items above, giving the estimated discharge at the point in question coming from the various subdivisions of the drainage area at different times. By tabulation of these rates of flow or by diagram, the time of the flood crest reaching any point and the maximum discharge can readily be ascertained.

In obtaining the time required for water to reach the given point on the river, the drainage area should be subdivided into several portions and the time for the water to reach the point in question estimated for each of these subdivisions. The velocity of flow in the tributary streams can be ascertained reasonably well by a few observations during ordinary high water, and also the approximate time required for surface flow to reach the smaller streams. Usually surface flow on lateral slopes will move with a velocity of from $\frac{1}{2}$ to 1 mile per hour; on the small branches the velocity may be from 2 to 4 miles per hour, depending on the amount of fall.

This procedure would not be required in such detail in the case of a large stream, but the same general method could be followed using larger subdivisions of time and rainfall data for 6-, 12-, and 24-hr. periods, etc.

The percentage of rainfall flowing off under flood conditions is likely to be from 50 to 75 per cent, although with conditions of sandy soil and flat slopes this may be considerably less. The selection of a suitable run-off coefficient may thus be somewhat uncertain in this method of flood estimate. On the other hand, data of maximum precipitation are often available where any other basis for flood estimates may be lacking, and this general method is, therefore, of value.

Run-off by the rational method is usually expressed by

$$Q = CiA \quad (23)$$

where C = a coefficient of ratio of run-off to rainfall.

i = intensity of rainfall in inches per hour, usually taken as the average rate during the concentration period, T_c in hours.

A = area in acres.

The run-off in inches is

$$R = CiT_c \text{ or } Ci = \frac{R}{T_c}$$

Hence

$$Q = \frac{R}{T_c} A$$

For similar drainage areas T_c will vary directly with the linear dimensions or directly with the square root of the drainage area. Hence, by changing the coefficient and making A square miles instead of acres, formula (23) becomes the same as (22):

$$Q = \frac{C_f AR}{\sqrt{A}} = C_f \sqrt{AR}.$$

CHAPTER III

THE STUDY OF STREAM-FLOW DATA AND WATER POWER ESTIMATES

The essential data of stream flow for water power studies have already been outlined on page 114, the necessary results being:

1. A flow-duration curve for the power site, to use as a basis for plant capacity and available power at all times. If storage is or is to be available at a site or sites upstream from the power plant, a flow-duration curve, as modified by the use of storage, will be necessary.

2. Low-water flow as a basis for available primary power and size of auxiliary, if used.

3. Flood flow to design properly the spillway of the dam and provide for the safety of the plant.

It is obvious that a daily record of flow of good length (10 years or more, at least) at the power site, or, in other words, flow data directly applicable, will provide for most of the above, with perhaps the exception of the extremes of flow—high and low. Even the latter may usually be predicted with fair accuracy from a 10-year record of flow, taking into account other information usually available in the way of precipitation data or flow of other streams, etc.

It is rarely true that adequate records of stream flow taken directly at the power site are available, and nearly always data at other sites or measuring stations on the river in question, or stations on some neighboring stream, must be used.

In the following discussion of methods of studying flow data in water power estimates, it will be assumed that data at the site, obtained either directly or indirectly, are available. The means employed for adjusting flow records for different sites on the same stream or as between different streams will be discussed later.

In studying data of stream flow for power estimates, it will generally be found that a combination of computations, usually in tabular form, together with a graphical study or plot of the same, will be most advantageous.

FLOW-DURATION CURVES

The flow-duration curve as ordinarily constructed consists of a plot of values of stream flow (daily, weekly, or monthly) in order of magnitude, as ordinates and per cent of time as abscissas, the curve thus showing the flow equaled or exceeded for any desired percentage of the time

covered by the record. (Note also that any function of discharge, such as horse power per foot of fall or for a given head, may also be used as ordinates for a duration curve.)

Methods of Plotting.—Two methods are in general use for constructing the flow-duration table, used for the plotting of the curve:

1. The calendar-year basis, where the records of each year are placed in order of magnitude and (using for convenience in illustration the month as a time unit) tabulated as the first, second, third, etc., month as regards dryness. There will then be as many lines of values in the table as there are years of record, and the mean of the first column will give the flow for the driest month of the average year, the mean of the second column that for the next driest month, etc. The result in this case will be a flow-duration curve formed by 12 points or months of the average year (*i.e.*, the average based on the number of years of record used).

If the week or day be used as a time unit, the method would be the same but the number of points on the curve would be correspondingly increased. A per cent of time scale can be plotted and used, if desired, to give abscissas in this unit instead of months or weeks, etc.

2. The total-period basis, where the values are placed in order of magnitude for the entire period, irrespective of the calendar year in which they occurred. To illustrate, for 10 years of monthly records there would be $10 \times 12 = 120$ points to form the curve, the lowest point of which might be, say for August, 1909, and the next value in magnitude that for September, 1908, etc.

In making this plot it is usually more convenient to adopt a scale of abscissas convenient for the time unit being used and when the plot is completed, by computation put on a decimal scale or one convenient for percentage use. The plot on this basis is often somewhat irregular in form and should usually be adjusted to a reasonably smooth curve.

This second or total-period basis gives more nearly a true percentage-flow curve, as not infrequently a year as a whole may be wet, with no months of really low flow. By the second method these relatively large values would appear where they belong in the curve; by the first method they might be averaged up with other values, often much less in amount, and tend to increase particularly the flow for the drier months of the average year. Similarly, the upper part of the curve by the first method would not reach such high limits as that by the second method. On Fig. 30 are shown for comparison flow-duration curves by each of these two methods.

While more laborious of application, especially with the day or week as a time unit, the second or total-period basis is, therefore, more accurate and generally preferable. Where a long record by days is to be plotted by the second method, it becomes necessary to follow some systematic method of arranging values by small groups in order of magnitude,

or, if preferred, each value may be written on a small card or slip of paper, which can be conveniently reassembled in order of magnitude and copied off in this order.

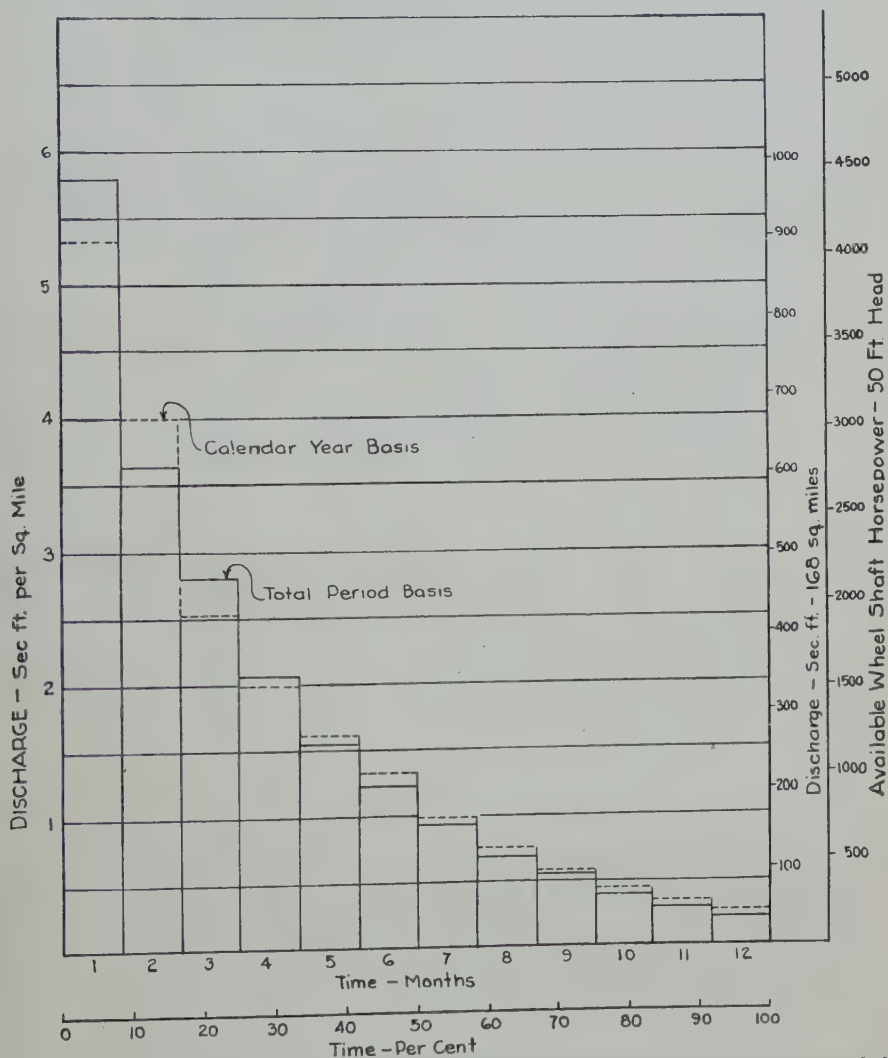


FIG. 30.—Flow-duration curves—Souhegan River at Merrimack N. H., for the period October, 1909, to September, 1923, inclusive, by months.

Effect of Time Unit on Form of Duration Curve.—Evidently a flow-duration curve based upon the day as a unit will give a curve more correct in detail, particularly for portions near each end of the curve, than one based upon the month as a unit, and, to a lesser extent, one based upon weeks as the unit of time. In Fig. 31 are shown for the Merrimack River

at Lawrence, Mass. (4452 sq. miles drainage area), flow-duration curves on both the monthly and weekly basis, and, as will be noted, in general the weekly curve is below the monthly for about 80 per cent of the

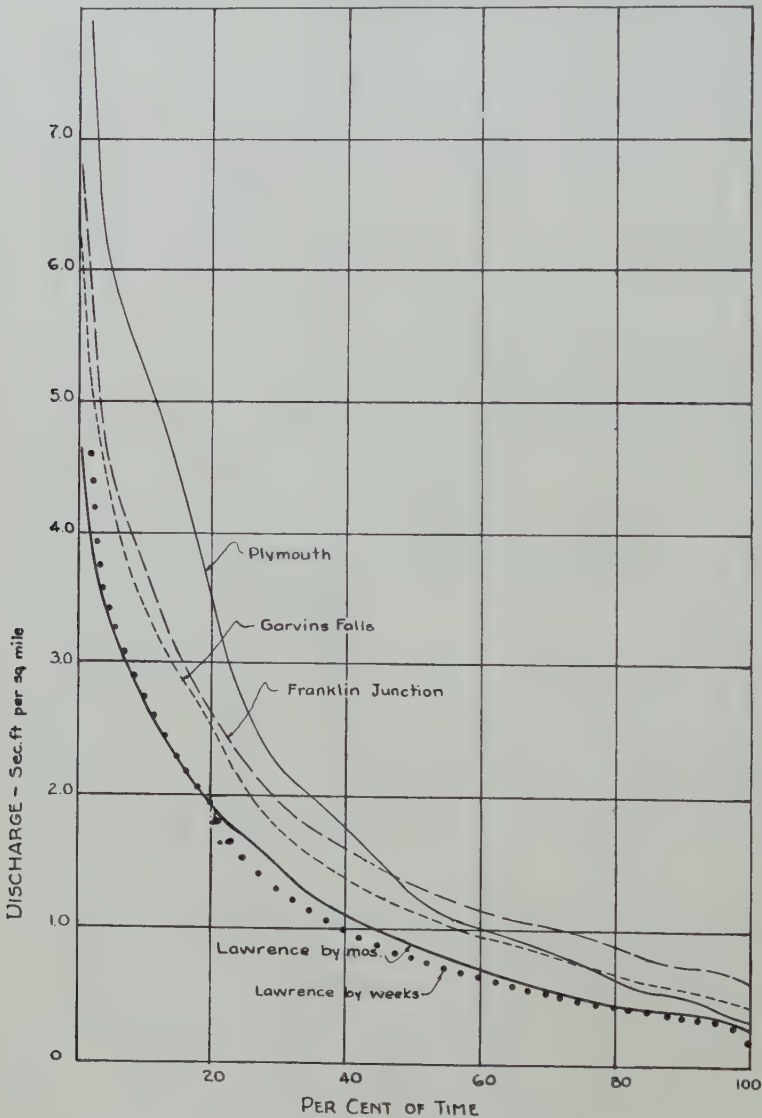


FIG. 31.—Flow-duration curves—Merrimack River.

time (the drier part of the year) and above for the rest of the year. In this case, however, the lower 80 per cent part of the weekly curve averages only about 3 per cent lower than the monthly curve.

In Fig. 31 are also shown flow-duration curves for three other stations on the Merrimack (or its tributaries) which going progressively northward are Garvins Falls (2340 sq. miles), Franklin Junction (1460 sq. miles), and Plymouth (615 sq. miles). The greater mean flow in the upper part of the basin (due essentially to greater rainfall in the more elevated portions) is obvious. The lower dry-weather flow at Plymouth is due to lack of storage on this tributary (the Pemigewasset), whereas the stations at Franklin Junction and below get the benefit of storage in Lake Winnepesaukee and other lakes.

Duration Curves as a Proportion of Mean Flow.—A useful form of flow-duration curve is that where ordinates are expressed as a ratio to

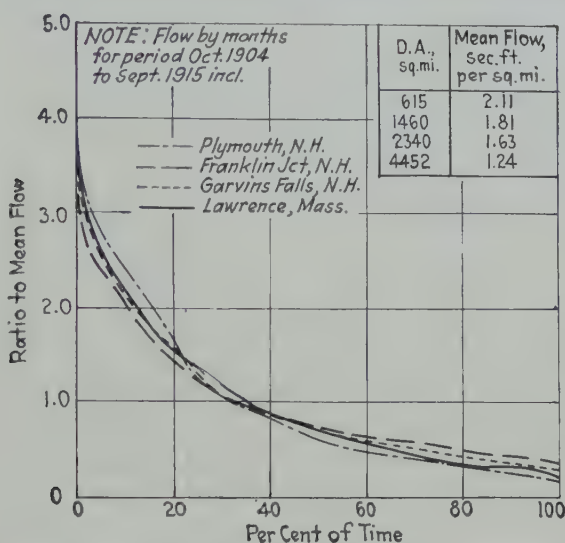


FIG. 32.—Flow-duration curves as a ratio to mean flow—Merrimack River.

the mean flow. The general form of this curve is often much the same even on different rivers, and it gives a means of approximating a flow-duration curve from the mean flow of a stream, which might be necessary, as in the case where little or no stream-flow records are available, but data of precipitation and of general conditions are at hand sufficient to establish within limits at least the mean yearly flow.

In Fig. 32 are shown flow-duration curves, plotted on the basis of ratio to mean flow, for the four stations on the Merrimack River basin, for which curves plotted in the usual manner appear in Fig. 31. As will be noted, the ratio basis gives curves much nearer alike in form than the unit-flow basis. By using a mean curve of those shown in Fig. 32, and applying ratios from it to the mean unit flow at any other station, a fairly good approximation of the usual flow-duration curve at the station may be obtained.

A study of flow-duration curves for numerous stations in different parts of the United States made under the direction of the author by A. S. Heyser in 1926, as a thesis at Massachusetts Institute of Technology, indicated that for rivers in the East and South the flow, as a per cent of the mean, could be expressed, for the drier 80 per cent of the time, by the formula $\log Q = 2.40 - 0.011 T$, where Q is discharge as a per cent of the mean and T is per cent of time. Further study by A. B. Daytz in 1928 indicates that the formula $\log Q = 2.45 - 0.011 T$ is more consistent as a general formula for streams in the East and South. These are of course average relations, and individual flow-duration curves in these districts will show variations.

A more extended study is needed to develop the full field of use of this type of flow-duration curve.

Duration Tables and Curves in Published Reports.—The U. S. Geological Survey in many of its *Water Supply Papers* gives tables of "Days of Deficiency of Discharge" and power arranged like the following example (see *Water Supply Paper* 424, page 35):

TABLE 42.—DAYS OF DEFICIENCY IN DISCHARGE OF OTTER CREEK AT MIDDLEBURY, VT. (615 SQ. MILES), DURING THE YEARS ENDING SEPT. 30, 1911–1916

Discharge, second-feet	Theoretical horse power per foot of fall	Days of deficiency in discharge					
		1910– 1911	1911– 1912	1912– 1913	1913– 1914	1914– 1915	1915– 1916
150	17.0	3	...	1	...	2	
200	22.7	8	3	14	4	21	
250	28.4	18	4	32	13	66	23
300	34.1	39	19	66	48	119	63
350	39.8	66	41	88	86	175	101
400	45.4	136	63	103	133	209	127
450	51.1	188	84	112	162	239	132
500	56.8	225	114	126	193	255	168
550	62.5	236	136	145	213	271	194
600	68.2	247	158	166	227	284	216
700	79.5	260	193	179	249	294	240
800	90.9	271	221	200	271	303	249
900	102	276	235	208	280	313	259
1000	114	284	243	220	290	314	271
1100	125	296	256	233	293	317	274
1200	136	303	263	244	300	320	287
1400	159	309	273	257	302	324	288
1600	182	310	283	270	309	331	294
1800	204	316	286	284	312	335	308
2000	227	327	296	301	318	342	315
2500	284	347	315	323	322	352	336
3000	341	358	338	350	326	361	355
4000	454	361	360	354	352	365	361
8000	909	...	366	365	365	...	366

The above table . . . shows the number of days on which the discharge and corresponding horse power were less than the amounts given in the columns for discharge and horse power.

This is a form of flow-duration table and may evidently be used to construct a flow-duration curve by summing for each discharge the days of deficiency, computing the per cent of this sum to the total number of days in the period covered by the table and plotting. Thus in the case of this table 150 sec.-ft. (or $\frac{150}{615} = 0.24$ sec.-ft. per square mile) was lacking 6 days out of $6 \times 365 = 2190$ days or about one-third of 1 per cent of the time; 200 sec.-ft. was lacking 50 days or 2.3 per cent of the time, etc.; and 250 sec.-ft., 156 days or 7.1 per cent of the time, etc. These three points on the flow-duration curve would be plotted as follows:

Per cent of time	Discharge available	
	Second-feet	Second-feet per square mile
100.0 ±	150	0.24
97.7	200	0.33
92.9	250	0.40

This method of obtaining the curve is analogous in results to the second method or total-period basis (page 126) and is also fairly convenient in preparation, as data for additional years may be added and revised results obtained with but little added computation.

In various state reports on water power (Maine, New Hampshire, and Massachusetts Water Power Commissions; New York Conservation Commission, etc.) will also be found flow-duration tables and curves, prepared as a rule on the total-period basis.

Effect of Storage on Form of Duration Curve.—It is evident that complete regulation by storage—if that were possible—would result in a duration curve in the form of a horizontal line corresponding to the mean flow.

With storage available to the usual extent, the effect is to raise the curve to the right of (for values below) the mean and to lower it at the left of the mean. As the mean point on the typical flow-duration curve is usually between the 30 and 40 per cent interval (*i.e.*, flow equaled or exceeded 30 to 40 per cent of the time), storage effect is to tend to raise the lower two-thirds of the curve and to lower the upper one-third.

This matter will be discussed later in more detail.

HYDROGRAPHS

A hydrograph may be defined as a plot of time units, as the hour, day, week, month, etc., in order of occurrence or calendar order, as

abscissas against some function of river discharge as ordinates. Thus as ordinates may be plotted:

- 1. River stage or gage heights.

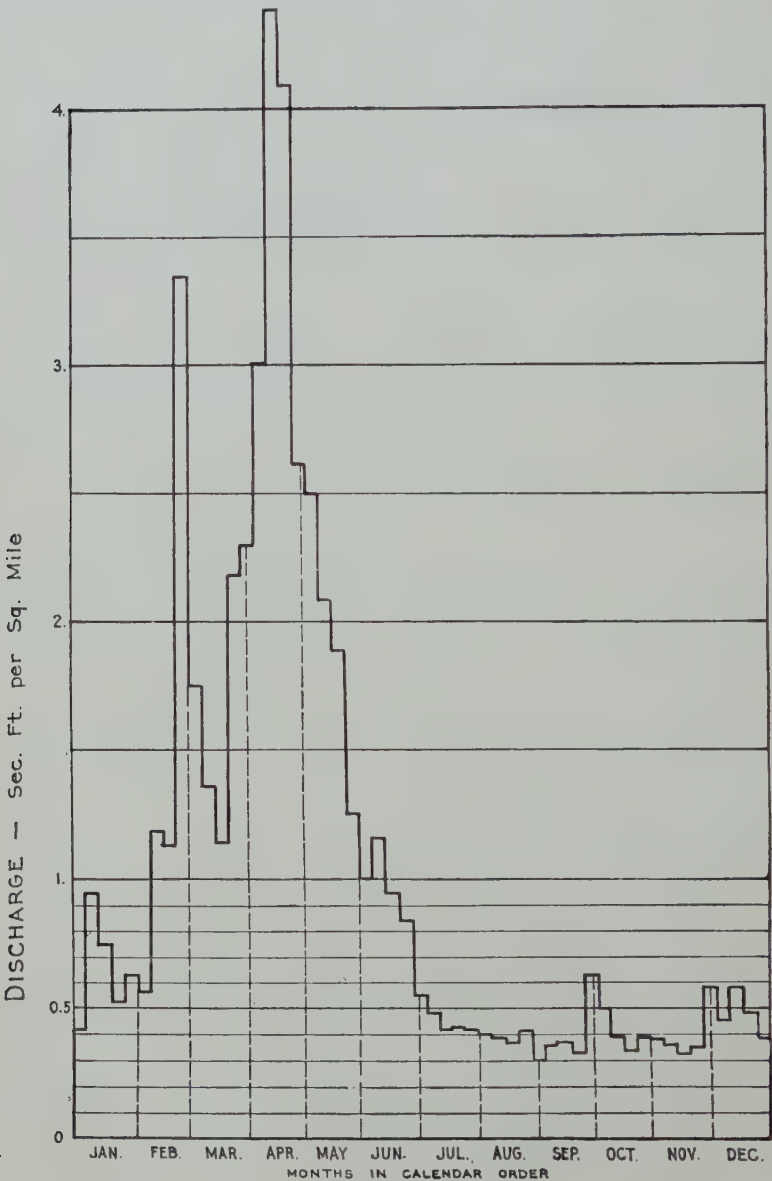


FIG. 33.—Hydrograph—Merrimack River at Lawrence, 1909.

- 2. Discharge in second-feet, or second-feet per square mile or inches depth per month, etc.

3. Horse power (theoretic or wheel shaft) per foot of fall or for some given head at a power site.

4. Horse-power days or horse-power months at wheel shaft for a given power site or kilowatt-hours at switchboard, etc.

The essential difference between hydrograph and duration curve, it will be seen, is that values are plotted in calendar order or actual order of occurrence for the former and in order of magnitude, irrespective of time of occurrence, for the latter.

The hydrograph shows, as it occurred, the variation in flow or available power of a stream and enables the extremes of flow to be more readily noted than does an inspection of the tabular values used in making it. The hydrograph is also essential in studies of the effect of storage on flow where obviously calendar order of discharge must be observed. On the other hand, the flow-duration curve for a period of years is in more convenient form than the hydrograph for the determination of proper wheel capacity and available power.

In Fig. 33 is shown a hydrograph for the Merrimack River at Lawrence for a single year (1909), based upon data of average weekly flow. A similar curve based upon daily instead of weekly flow would evidently show considerably greater variation, while a hydrograph of monthly mean discharge alone would be of less variation than this which is based upon weekly flows.

In Fig. 34 are shown for comparison the same data as on the hydrograph (Fig. 33) arranged in the form of a flow-duration curve.

While in the case of both Figs. 33 and 34 the unit of discharge is taken as the second-foot per square mile, by placing one or more auxiliary scales at either the left or right of the plot, values for other functions of discharge and horse power may be readily taken from the plot.

STUDIES OF STORAGE EFFECT

The two general effects of storage upon stream flow have already been pointed out, *viz.*: (1) the tendency to equalize flow at different times of the year, which is of course the fundamental purpose of storage; and (2) the lessening of the average flow of the stream (commonly), due to the increased evaporation from the water area created by the reservoir, as compared with that from land area.

Methods of Operating Storage.—While the operation of storage consists in general in holding back in reservoirs the excess flow available in times of high water (most often the spring months in the North) and releasing it more or less regularly during the dry season, we may distinguish two fairly distinct extremes in operating, *viz.*: (1) the yearly-use method, where it is intended practically to empty the reservoir at the end of each (water) year, and not, as a rule, to carry over any considerable

amount of stored water for the next season; and (2) the safe-yield or best-dependable-flow method, this flow having been approximately

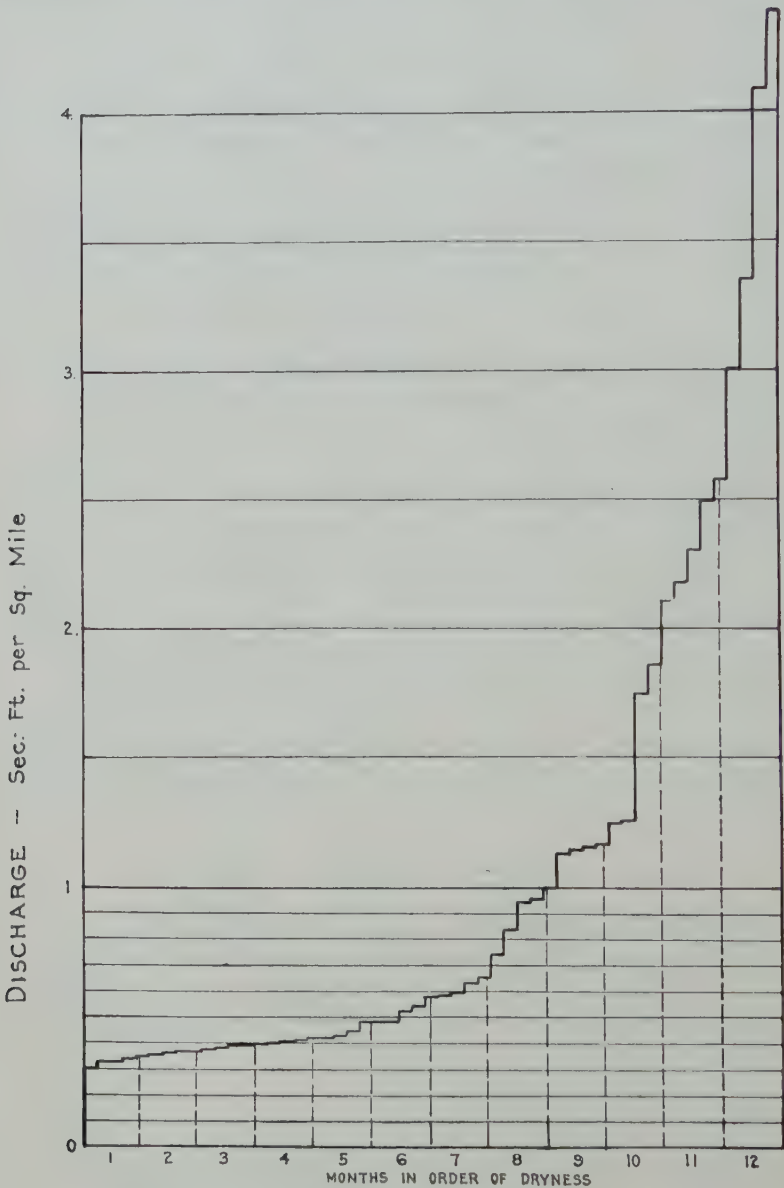


FIG. 34.—Flow-duration curve, Merrimack River at Lawrence, 1909.

determined by a study of previous dry periods, and the rule adopted in operating not to draw on the reservoir capacity at any time to exceed this safe yield. This method usually involves holding stored water over

more than one season, perhaps several seasons in the case of high rates of storage development. It gives a maximum of primary power from storage but also results in more waste of water over the reservoir spillway and a less total available power output in kilowatt-hours per year from stored water than the first method.

Most storage operating for power purposes is, therefore, more in accordance with the first or yearly-use method, especially where it is possible to safeguard primary power by interconnection with other plants, especially with steam plants.

The use of the Somerset reservoir on the Deerfield River is a good illustration of operating storage by the yearly-use method, while the operation of the storage reservoirs in the headwaters of the Androscoggin River, Maine, including the Rangeley Lake system, the Azischohos reservoir, etc., is more essentially by the second method, *viz.*, to insure a minimum flow of the river at Berlin, N. H., of not less than 1550 sec.-ft.

The problem of storage operation, particularly where numerous power plants at some distance apart are to be served, is often a complicated one. It is usually impossible to forecast with any accuracy the general conditions of rainfall and water supply that will prevail in a given season, and storage operation must be based upon a study of past performance, as shown by the record of yield of the stream, combined with good judgment, all aided by the experience previously gained in operating a particular river.

Where the chief water supply for reservoirs is from snow storage, *i.e.*, the accumulation of a winter's snow, as it occurs in many of our northern states, it is often possible, however, to forecast with some accuracy the probable available water supply at the beginning of the storage-use season by making approximate "snow surveys"; *i.e.*, determining the depth of snow on the ground and its water equivalent, and applying a run-off factor based upon previous run-off records, of course making proper allowance for rains normally likely to occur up to about May or June. This method is in very practical use on several streams in New England where storage is made use of for power.

The study of the effect of storage regulation based upon previous records of yield of a stream is of great importance, both in aiding a decision as to the desirability and amount of storage and its value, as well as for use as a guide in future operating, in case the reservoir is constructed. Various methods of making such studies will now be discussed, preceded by an outline of these methods and the problems to which they may be advantageously applied.

Methods of Studying the Effect of Storage.—Two general cases present themselves in respect to storage location, *viz.*:

1. Storage reservoir at the power development, so that the problem involved is the flow obtainable from and at the reservoir.

2. Storage reservoir some distance above the power plant, so that an intervening uncontrolled area (*i.e.*, with reference to storage) should be taken into account in determining the flow obtainable at the power plant.

Various complications of these two simple cases may occur. More than one reservoir may be planned or used, and ordinarily there are a number of power plants at various sites along the river below the reservoirs, often many miles distant with large intervening uncontrolled drainage areas. Usually some simple assumption in regard to the total amount of storage and its effective or virtual location can be made while operation may be planned for the "center of gravity" of the power system, enabling an approximate solution of the problem of best use of water, which can be further checked by more detailed computations for each power plant. The center of gravity of power may be based on the factor, drainage area $\times$ head developed (or available head) or on actual wheel capacity in the case of a fully developed power system.

For the first general case, where the power development is at the reservoir, the mass-curve method of study will be found convenient; for the second or more general case, the hydrograph method, with accompanying reservoir-depletion curve, is best adapted.

Mass-curve Method for Studying Effect of Storage.—This useful graphical method was first suggested by Rippl,¹ a British engineer, for the study of effect of storage on flow in problems of water supply and is, therefore, often called the Rippl method.

It consists in graphically representing the total amount of water available at any time for storage during the period in question, this being the flow of the river into the storage reservoir, with an allowance for any loss occasioned by storage, such as evaporation, seepage, etc., or, if some fixed amount of water is to be diverted from the drainage for any reason, this will also figure as a loss. The net quantity of water remaining is available for power purposes, and these amounts by months are added one to the other consecutively and plotted in a curve in which the abscissa of each point represents the total time from the beginning of the period and the ordinate the total quantity of water available during this time. The time scale would usually be months (more rarely weeks or years), while the water scale may be inches depth on the drainage area, cubic feet, acre-feet, or second-foot months. The latter (1 sec.-ft. maintained for a month) is convenient as relating at once the unit of flow and of volume (see Fig. 35).

The inclination of the curve at any point indicates the rate of the net flow at that particular time. If the losses due to evaporation, seepage, etc., are small so that they never are the equivalent of a single month's supply, the curve will evidently always have a positive slope but of a greater or less degree, depending on the variation in the quantity of water

¹ *Proc. Inst. Civil Eng.*, vol. 71, p. 270.

available. A negative inclination of the supply line would show that the amount of water flowing into the reservoir was less than the loss due to evaporation, seepage, etc. The curve of demand for use of water can be plotted usually as a straight line, and lines corresponding to different rates of draft are shown on the figure.

The ordinate between the supply and any one of these demand lines represents the total surplus from the beginning of the period considered, and, when the inclination of the supply line is less than that of the demand line, the yield of the drainage area is less than the demand, and water

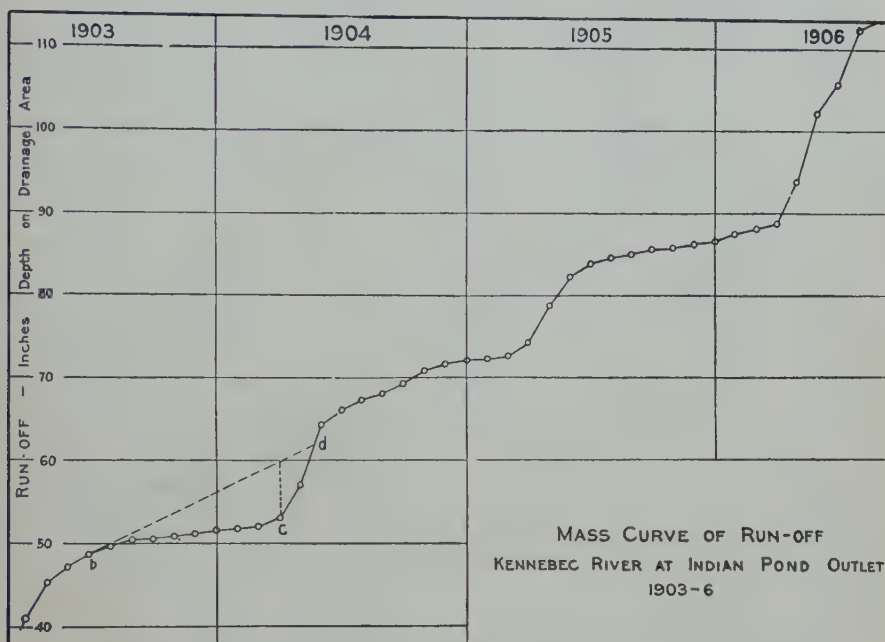


FIG. 35.—Mass curve for studying storage effect.

must be stored. The amount of deficiency during the dry period is found by drawing lines parallel to the demand line and tangent to the supply curve at its various summits, as at *b* (Fig. 35). The maximum deficiency in the supply and necessary reservoir capacity to maintain the demand during period of draft is shown by the maximum vertical distance between the tangent to the mass curve and the mass curve. The period during which the reservoir will be drawn below the high-water line is, say, from *b* to *c*; the reservoir would be filling from *c* to *d*, and water would be wasted from *d* on for some time.

If the tangent from any summit parallel to any demand line fails to intersect the curve, it means that during this period the supply is inadequate for the demand, or, in other words, to insure a full reservoir a

parallel tangent drawn backward from the low points on the curve must intersect the curve at some previous point.

The mass curve should not generally be used by scaling results alone but rather by making analytical computations using the table of values from which the curve is plotted and accurately computing any desired ordinates, using the plot as a check by scaling. The method of computing these ordinates will be evident with a little thought.

Aside from checking results, the plot is useful and in fact necessary to insure that all possible periods of drought are investigated and in general to make clear the method to be followed in the computations.

Another form of mass curve sometimes used is the residual-mass curve, which is a plot of time against total available water supply, after taking out the assumed use of water during the month as well as any losses due to seepage, etc. Evidently this curve will show surplus water available for storage while rising and deficient water or that which must come from storage when the curve is falling. Storage required will thus be the ordinate between a horizontal line drawn from any given summit of the curve and an ensuing valley point. This form of mass curve is evidently more limited in its scope of use than the first type.

Storage-draft Curves.—A very useful plot of results from a mass-curve study of storage may be made by plotting, say, storage required as abscissas and corresponding rates of draft as ordinates, such a curve enabling at once the determination of the possible dependable rate of flow for any assumed amount of storage. To be more generally useful, unit rates of draft (second-feet per square mile, inches depth per month, etc.) may be used with necessary storage for an area of, say, 1 sq. mile (or often 100 sq. miles, for convenience).

The storage-draft curve will evidently be a series of straight lines meeting in angles, each angle caused by any one of the following situations on the mass curve proper:

1. A change in position of the ordinate of storage required in a given valley of the curve.
2. A change of the tangent point or initial point of the draft line.
3. A complete change of valley for the storage required ordinate.

To make the storage-draft curve more generally useful, it may be adjusted by computation (or further mass-curve studies) to show curves for different percentages of water area, as 0 (land area alone), 5, 10 per cent, etc. The curves for the greater percentages of water area will evidently lie to the right of the others, *i.e.*, more storage is required to maintain any given rate of draft with larger water area because of the greater water losses of evaporation from reservoir surface.

In Fig. 36 is shown a mass-curve study of storage effect, with resulting storage-draft curves (1) for best primary output and (2) for yearly use of storage, for the Raquette River at Piercefield, N. Y., for the 15-year

period 1908-1922. The draft lines shown in connection with the mass curve up to about 1.29 sec.-ft. per square mile are applicable to either method of storage operation. For yearly use of storage no greater rate

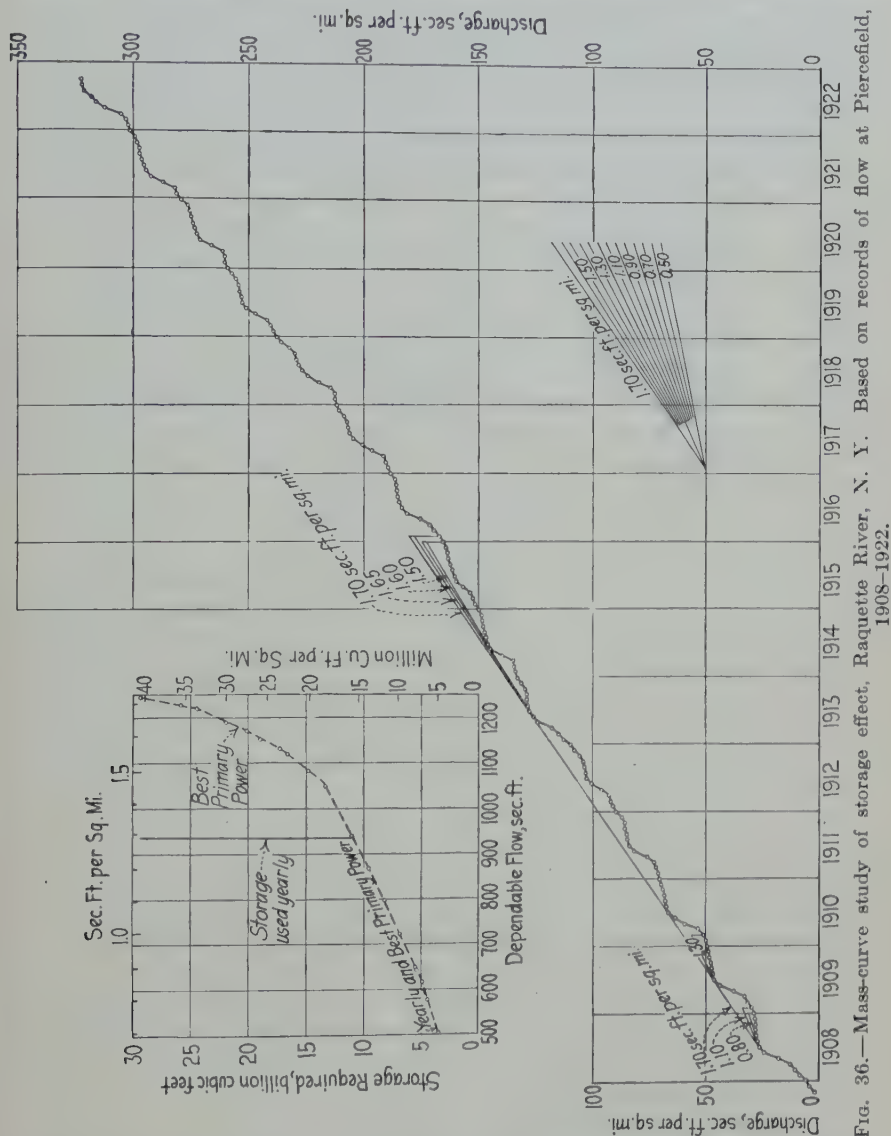


Fig. 36.—Mass-curve study of storage effect, Raquette River, N. Y. Based on records of flow at Piercesfield, 1908-1922.

of draft than 1.29 can be obtained by additional storage capacity due to lack of water for storage in the spring of 1915. Hence, beyond this rate of draft in this case the storage-draft curve is a vertical line as shown.

With storage operated on the best primary-power basis the gain in dependable flow over and above about 1230 sec.-ft. (1.70 per square

mile) due to increased storage capacity is slight, as this is approaching the average flow for the 15-year period, of about 1300 sec.-ft. (1.80 sec.-ft. per square mile).

Hydrograph Method for Study of Storage Effect.—As previously stated, the hydrograph method is better adapted than the mass curve to study the effect of storage in the more general case where the power plant (or plants) is at some distance from the reservoir, with considerable intervening uncontrolled drainage area. Some simple cases of a single reservoir at the head of the river and a single power plant will first be considered.

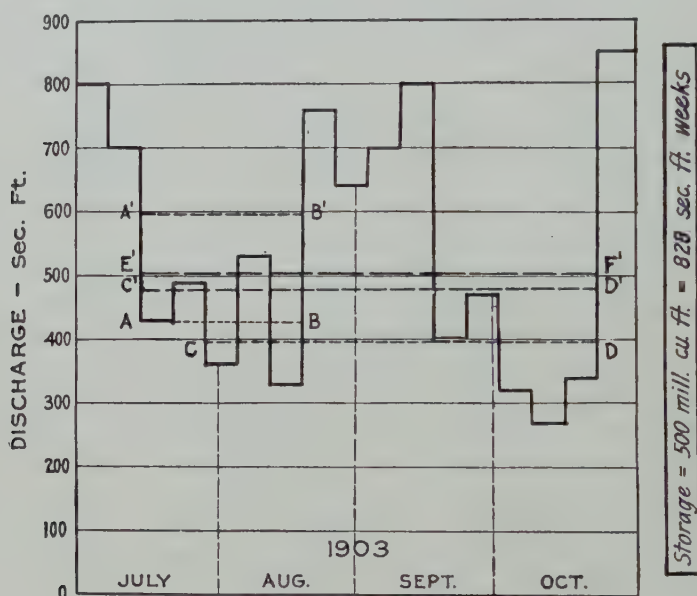


FIG. 37.—Hydrograph study—limited storage.

Limited Amount of Storage.—In Fig. 37 is a hydrograph study of storage effect using the week as time unit, for the following assumed conditions:

Reservoir: Storage capacity 500 mill. cu. ft. or 828 sec.-ft. weeks at head of river; water area 2 sq. miles; total drainage area of reservoir 20 sq. miles.

Power Plant: Total drainage area 300 sq. miles, of which 20 sq. miles is controlled by the reservoir.

The available storage capacity of 500 mill. cu. ft. for a total drainage area of 300 sq. miles or about 1.67 mill. cu. ft. per square mile is small and will be of service only for a short time each season during the dry period, although with so little storage capacity it may be possible to use this more than once during the season. Conditions during the dry

period of a single season (1903) are shown to illustrate possible methods of storage operation, assuming, as would nearly always be the case with such limited storage, that the reservoir is full at the beginning of the dry period.

The hydrograph is plotted for the uncontrolled flow from the area, below the reservoir (*i.e.*, 280 sq. miles), the problem being how best to fill in the valleys or gaps in this hydrograph by drawing on storage. In such hydrograph studies there are three elements contributing to the regulated flow during the dry period, *viz.*:

1. The flow from the uncontrolled portion of the drainage area below the reservoir. This is a foundation element, which cannot be changed as to manner of occurrence.

2. Water stored in the reservoir at the beginning of the dry period, under control at all times.

3. The net flow available during the dry period from the reservoir drainage area, consisting (*a*) of the yield of its land area and (*b*) of the yield of its water area, this last usually being negative whenever evaporation exceeds rainfall on the reservoir, which is commonly the case during the dry season. This third element is, as a rule, under control, like the second.

Considering the first dry period from about July 15 to Aug. 20, AB is the average uncontrolled flow, which may normally be used as a basis for the built-up hydrograph. (The exception would be when some week near the end of the period was materially higher than any of the others, so that its inclusion in computing the mean would result in assuming that some of the flow would have to be used prior to the week when it occurred—an impossibility.)

If storage were all used during this first dry period of the summer, a flow of $A'B'$ might be obtained. Obviously, however, during Sept. 15 to about Oct. 20 the possible regulated flow (not shown) would fall much below the level of $A'B'$. A much better dependable or primary power flow could be obtained by considering the whole period from July 15 to Oct. 20, for which CD is the first element or average available uncontrolled flow, $C'D'$ is the extent to which flow could be built up by stored water which was on hand July 15 (the second element), and $E'F'$ the true dependable flow taking into account the yield of the reservoir drainage area during the dry period (the third element).

Before taking this as the final or correct result, it would be necessary, however, to construct a reservoir-depletion curve or table (not here shown), and check up week by week to make sure that in no case the use of the average figures involves any error of impossible use of water as explained previously.

Larger Amount of Storage.—Figure 38 is a hydrograph study by months of the effect of a larger amount of storage, for the following assumed conditions:

Reservoir: Storage capacity of 3 bill. cu. ft. or 1140 sec.-ft. months at head of river; water area 4 sq. miles; total drainage area of reservoir 80 sq. miles.

Power Plant: Total drainage area 880 sq. miles, of which 80 sq. miles is controlled by the reservoir.

Storage per square mile of total drainage area is about 3.4 mill. cu. ft., enough to enable a fair degree of regulation of flow, but by no means complete.

It will be further assumed that it is desired to operate to give the best output of primary power.

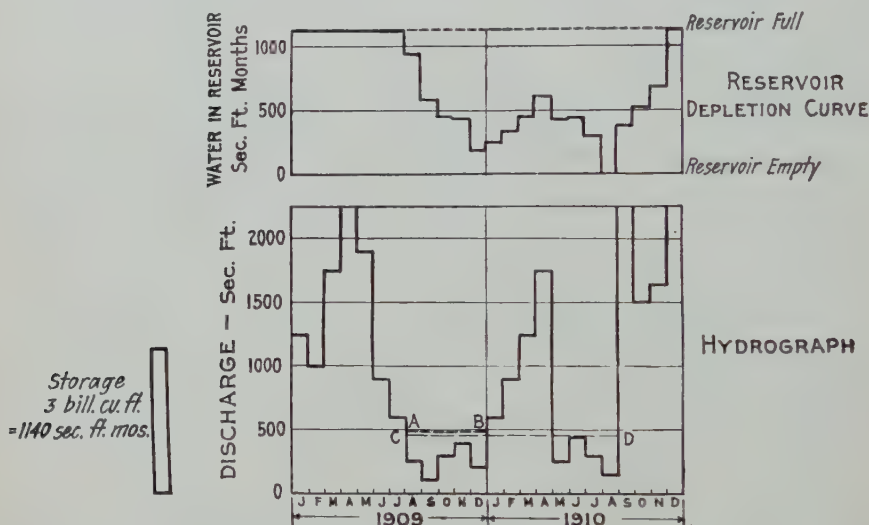


FIG. 38.—Hydrograph study—large storage.

In Table 43 appended are the computations in tabular form accompanying Fig. 38 which may be analyzed as follows:

In col. 2 are the items which make up the first general element, of uncontrolled flow.

At the head of col. 11 is the second element, of water in reservoir, at the start of 1140 sec.-ft. months.

In cols. 3 to 8, inclusive, are the details of estimate of the third element, *viz.*, the yield of the 80 sq. miles of controlled area during the period studied.

Anticipating the results of the computation for the moment, *CD* on the hydrograph, corresponding to 458 sec.-ft. in col. 9 of the table, is the dependable flow that can be exactly maintained through this entire period of drought. It is not known in advance, however, just what period will be included by *CD*, in other words, just how many months are to be considered in the computation, and it is usually necessary, therefore, to

assume this period of months, make a trial computation for *CD*, and repeat as necessary until the sum of the three elements available for filling the hollows in the hydrograph corresponds to the period of months considered.

Thus in this case the period in question is 13 months, whereas in the trial computation 15 months may have been first assumed, not knowing to just what extent the hydrograph might be filled up.

The computation when corrected would be as follows:

First Element	Second Element	Third Element	Total
Aug. 1909..... 250	Aug. 1909..... 1140	Aug. 1909..... 9	
			1
			21
			35
Dec. 1909..... 200			17
May 1910..... 250			61
			92
			125
Aug. 1910..... 150			170
			12
			28
			12
		Aug. 1910.....	-2
2400	1140	581	4121

$$412\frac{1}{9} = 458 \text{ sec.-ft.} = \text{dependable flow.}$$

This figure of 458 must be checked, however, as previously noted, by computing for each month (col. 11) water in reservoir as a basis for the reservoir-depletion curve shown on Fig. 38. The test of the computation is the (substantial) emptying of the reservoir at the end of the period considered.

If these computations had been made on the basis of yearly use of storage instead of that for best primary power, evidently a higher rate of flow than 458 sec.-ft. could have been maintained in 1909, but as the reservoir would not have filled in the spring of 1910 the regulated flow during the summer of 1910 would have fallen much below 458 sec.-ft.

The student should, in accordance with the yearly-use method, make a revised table and hydrograph, assuming the end of the 1909 water-year as, say, Dec. 30, 1909, and compare the results with those given in Table 43.

Effect of Storage—Two or More Power Plants.—With a single reservoir assumed as before but two power plants some distance down river, the first essential is to determine for which plant regulation of flow is to be made. This may often be settled by the relative size of the plants, the point being to choose a manner of regulation that will give a maximum power output from the combined plants.

TABLE 43.—USE OF HYDROGRAPH
(Tabular computations for Fig. 38)

Year and month	Discharge 800 sq. miles, second-foot	Reservoir						Assumed discharge at plant, second-foot	Drawn from reservoir, second-foot	Left in reservoir, second-foot months
		Discharge 76 sq. miles, second-foot	Precipita- tion, inches	Evapora- tion, inches	Precipitation minus evapo- ration		Net yield 80 sq. miles, second-foot			
					Inches	Second-foot 4 sq. miles				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1909:										
July.....	600	57	2.24	5.98	-3.74	-13	44	644	0	1140
Aug.....	250	24	1.30	5.50	-4.20	-15	9	458	208	941
Sept.....	100	10	1.50	4.12	-2.62	-9	1	458	358	584
Oct.....	300	28	1.23	3.16	-1.93	-7	21	458	158	447
Nov.....	400	38	1.50	2.25	-0.75	-3	35	458	58	424
Dec.....	200	19	1.00	1.51	-0.51	-2	17	458	258	183
1910:										
Jan.....	600	57	2.00	0.96	+1.04	+4	61	600	0	244
Feb.....	900	86	2.80	1.05	+1.75	+6	92	900	0	336
Mar.....	1250	119	3.30	1.70	+1.60	+6	125	1250	0	461
Apr.....	1750	166	4.00	2.97	+1.03	+4	170	1750	0	631
May.....	250	24	1.00	4.46	-3.46	-12	12	458	208	435
June.....	450	43	1.20	5.54	-4.34	-15	28	458	8	455
July.....	300	28	1.50	5.98	-4.48	-16	12	458	158	309
Aug.....	150	14	1.00	5.50	-4.50	-16	-2	458	308	-1
Sept.....	4000	380	8.40	4.12	+4.28	+15	395	4000	0	395
Oct.....	1500	142	4.00	3.16	+0.84	+3	145	1500	0	540
Nov.....	1650	157	6.40	2.25	+4.15	+15	172	1650	0	712
Dec.....	8400	800	9.80	1.51	+8.29	+29	829	8400	0	1140

Once the point on the river is established for which regulation of flow is to be made, the results at this point may be worked out by a hydrograph study in the manner previously described. The continuation of this study to obtain the modified flow at a plant farther downstream will evidently be a simple matter, usually requiring only the addition of a certain amount of (uncontrolled) flow from the intervening drainage area.

With more than two plants the general procedure would be similar, first to decide for what point regulation is to be directed—often the center of gravity of power, as previously explained—work out the available flow at this point consistent with this regulation, by hydrograph study, and follow this by determining the corresponding flow at the other plants or power sites.

With two or more reservoirs the problem becomes more complicated, especially with some of the power plants between the reservoirs. The same general method of solution by hydrograph study may be used, however, and often it will help to consider the entire drainage area above the lowest reservoir to be controlled (or largely so) by the total reservoir capacity, considered first as a unit and then in detail with particular reference to the results obtainable at any plants between the reservoirs.

Thus on the Deerfield River with the Davis Bridge reservoir recently completed, there is available for storage:

TABLE 44.—DEERFIELD RIVER STORAGE

Reservoir	Capacity, million cubic feet	Total drainage area at reservoir, square miles	Storage capacity		
			Million cubic feet per square mile	Inches depth on drainage area	Per cent of yearly run-off = 32 in.
Somerset.....	2700	30.4	89	38.0	119
Davis Bridge.....	5000	184	27	11.7	36
Total.....	7700	184	41.8	18.2	57

Evidently from the above table, as far as results at and below Davis Bridge reservoir are concerned, the total storage of 7700 mill. cu. ft. may be looked upon as in one reservoir, at Davis Bridge, and the operating study made on this basis. The further study must then be made of operating the Somerset reservoir to give best results at the Searsburg plant and any other plants which may be built between these two reservoirs.

An example of the results of a study by the writer in 1918-1920 of storage effect upon the Kennebec River, Maine, is given in Figs. 39 and 40. In this case a total storage capacity of 47 bill. cu. ft. in the upper portion of the river was assumed to be utilized for best power output at Bingham, the approximate center of gravity of power on the main river, on which a total fall of about 882 ft. exists, with 197 ft. now developed. Hydrograph studies were made for the period 1902-1916 on the basis

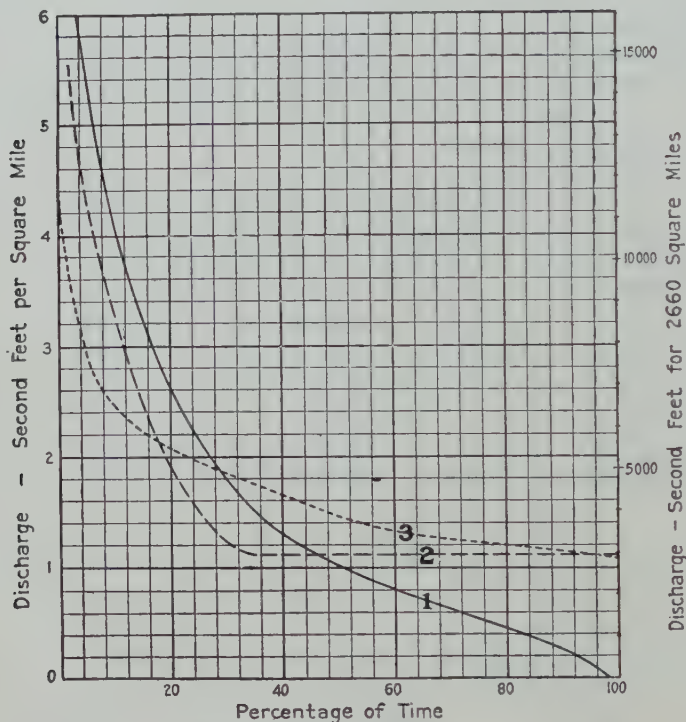


FIG. 39.—Flow-duration curves for Kennebec River at Bingham.

1. Flow corrected for storage.
2. Flow as modified by storage (47 bill. cu. ft.), utilized for best primary power at Bingham.
3. Flow as modified by storage, as in 2, used yearly.

of (1) regulation for best primary-power output and (2) yearly storage use, and from the results of the hydrograph study the modified flow-duration curves in Figs. 39 and 40 were obtained. On these diagrams are also shown the "flow corrected for storage," *viz.*, as it would have occurred with no storage regulation, and in the case of Waterville (Fig. 40) "the flow as observed," which was affected by some storage regulation, chiefly in Moosehead Lake.

A flow-duration curve for one other point on the river upstream from Bingham (not shown) was also made. These three curves, with a

consideration of intervening drainage areas, permitted the estimate of power, both primary and secondary, as affected by storage at each site on the river. In concluding as to the effect of storage, a mean of the values given by the two sets of curves, *viz.*, for best primary output and yearly use, was adopted as representing the most probable result of actual storage operation.

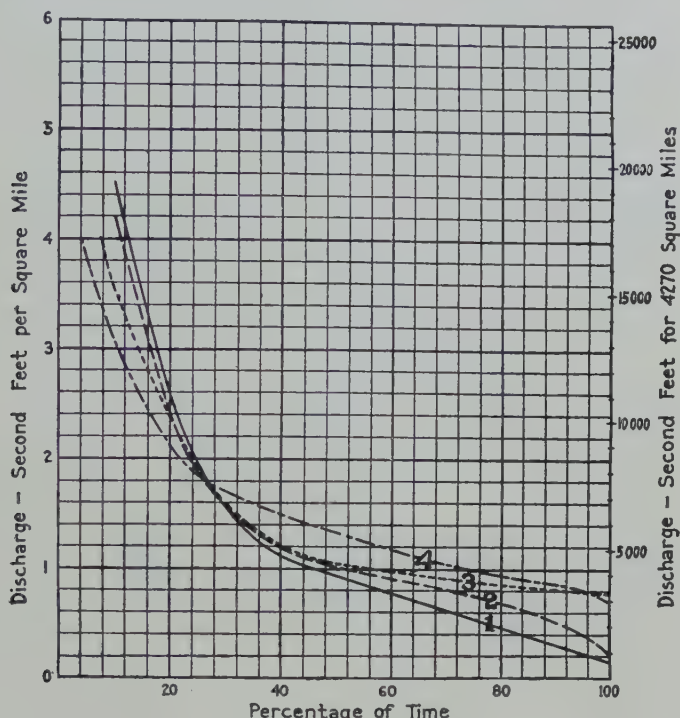


FIG. 40.—Flow-duration curves for Kennebec River at Waterville.

1. Flow corrected for storage.
2. Flow as observed.
3. Flow as modified by storage (47 bill. cu. ft.), utilized for best primary power at Bingham.
4. Flow as modified by storage as in 3, used yearly.

Storage Effect—General Studies.—Where records are not available on a river to make a hydrograph study of storage effect, a general method, based upon results from other rivers, may be used, as was done by the author in determining the effect of proposed reservoirs upon power output and flood prevention in Vermont.¹

This consists (see Fig. 41) of plotting (horizontally) storage as a ratio of mean yearly run-off against (vertically) (a) maximum (95 per cent) flow and (b) average flow up to wheel capacity, both expressed as a ratio to the mean yearly flow.

¹ Report of consulting engineer to Advisory Committee of Engineers on Flood Control, Vermont, December, 1931, Fig. 24, opposite p. 119.

The individual curves shown on Fig. 41 were based upon hydrograph studies of storage effect on various rivers with different assumed amounts of storage per square mile of drainage area.

While there is some variation in the individual curves, the "mean curve" affords a reasonably good basis for storage-effect determinations

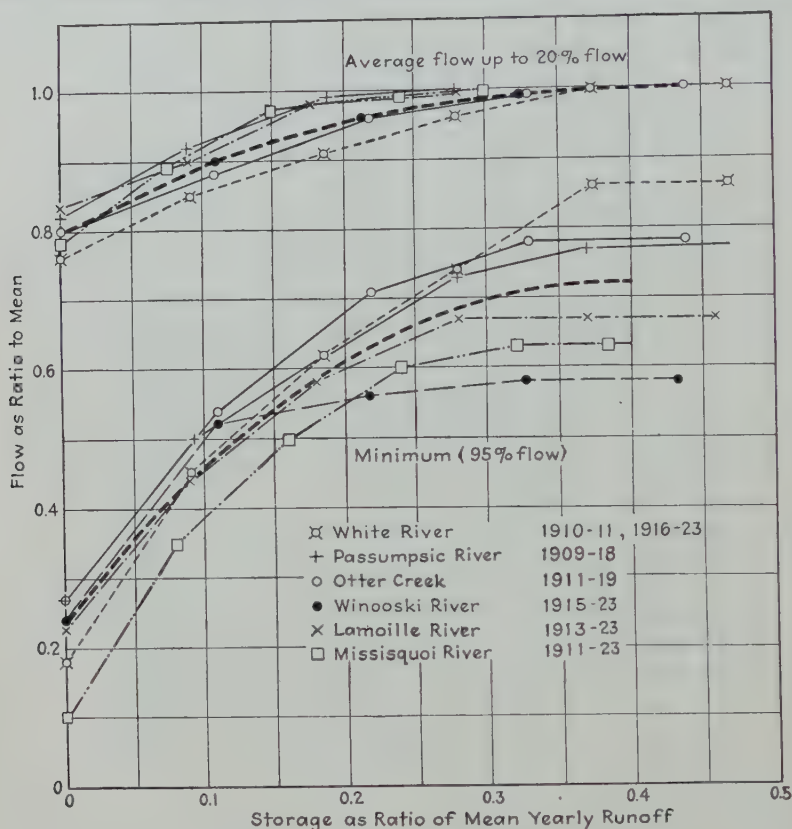


FIG. 41.—Storage—flow relations for Vermont rivers.

upon rivers in the general vicinity where no flow records are available. The use of these curves will be evident from the following example:

Black River, Vermont—Ludlow Reservoir Site

Drainage area	= 56 sq. miles
Mean yearly run-off	= 1.77 sec.-ft. per square mile
Available storage	= 1.2 bill. cu. ft.
	= 21.4 mill. cu. ft. per square mile
	= 9.3 in. depth
	= 0.39 of mean yearly run-off

From Fig. 41, mean curve (heavy dotted lines)

Minimum (95 per cent) flow	= $0.72 \times 1.77 \times 56 = 76$ sec.-ft.
Average (up to 20 per cent) flow	= $1.00 \times 1.77 \times 56 = 100$ sec.-ft.

Storage Capacity as Developed.—In Table 45 are given details of various storage reservoirs for power use as constructed; also data for certain proposed reservoirs, some of which are under construction. In addition to the capacity in million cubic feet, that expressed as inches depth on the tributary drainage area is also given. The mean yearly run-off in inches depth is also given and the proportion of this which corresponds to the reservoir capacity.

TABLE 45.—STORAGE RESERVOIRS FOR POWER USE—CAPACITY
(Constructed Except as Noted)

Reservoir	Location, river and state	Drainage area, square miles	Capacity		Inches depth on drainage area	Mean yearly run-off, inches	Ratio 6:7
			Million cubic feet				
			Total	Per square mile			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1 Azisecos.....	Magalloway, Maine	233	9,600	41.4	17.8	22	0.81
2 Ripogenus.....	Penobscot, Maine	1,410	30,000	21.3	9.3	23	0.40
3 Somerset.....	Deerfield, Vermont	30.4	2,700	89.0	38.0	32	1.19
4 Davis Bridge.....	Deerfield, Vermont	154	5,000	32.5	14.0	32	0.45
5 (3 and 4).....	Deerfield, Vermont	184	7,700	41.8	18.2	32	0.57
6 Indian Lake.....	Indian, New York	132	5,000	37.8	16.3	28	0.58
7 Burton.....	Tallulah, Georgia	136	5,280	38.8	16.8	40	0.42
8 Matthis-Tallulah..	Tallulah, Georgia	17	1,369	80.5	34.7	40	0.87
9 (7 and 8).....	Tallulah, Georgia	153	6,649	43.5	18.7	40	0.47
10 Lake Almanor....	North Fork Feather, California	479	57,000	119.	51.2	30	1.71
Southern California Edison Company System	San Joaquin and Bear Creek, California						
11 Present.....	Two reservoirs (Huntington and Shaw lakes)	448	4,100	9.2	4.0	15 ±	0.27
12 Ultimate proposed	Twelve reservoirs Moosehead Lake and above	1,200 ±	32,000	26.6	11.5	15 ±	0.77
13 Present.....	Chiefly Moosehead Lake	1,240	26,000	21.0	9.1	24	0.38
14 Ultimate proposed	Two additional reservoirs, etc.	1,240	46,800	37.6	16.3	24	0.68
15 Sacandaga.....	Hudson, New York	1,044	30,000	28.7	12.3	26	0.47
16 Bagnall.....	Osage, Missouri	14,000	52,300	3.8	1.6	10	0.17
17 Hiram (proposed).	Saco, Maine	800	10,000	12.5	5.4	25	0.22
18 Oxbow (proposed).	Raquette, New York	498	15,000	30.0	13.0	23	0.57
19 La Loutre.....	St. Maurice, Quebec	3,650	160,000	43.8	18.9	17	1.11
20 Kenogami.....	Saguenay, Quebec	1,400	13,000	9.3	4.0	23	0.17
21 Lake St. Francis...	St. Francis, Quebec	472	12,200	25.8	11.2	26	0.43
22 Boulder Dam.....	Colorado, Arizona-New Mexico	170,000	830,000(a)	4.9	2.1	1.7	1.24

(a) Between elevation 1229 and 1050 ft.

TABLE 46.—USE OF STORAGE CAPACITY, AVERAGE YEAR

River and state	Time	Reservoir site			Regulation point		Storage capacity, inches depth for drainage area		Mean yearly run-off, inches	Proportion of storage used	
		Storage capacity, million cubic feet			Location	Drainage area, square miles	At reservoir	At regulation point		(1) Yearly use of storage	(2) Safe yield method of use
		Drainage area, square miles	Total	Per square mile							
Based on Storage Studies											
Kennebec, Maine.....	1902-1916	1,240	47,000	37.8	Bingham	2,660	16.2	7.9	24.8	0.75	0.50
Saco, Maine.....	1908-1916	800	10,000	12.5	West Buxton	1,550	5.4	3.9	26.0	0.87	0.55
Hudson, New York.....	1900-1916	1,044	34,600	33.3	Spier Falls	2,777	14.3	5.4	26.8	0.87	
Raquette, New York.....	1904-1922	498	16,000	32.0	Piercefield	723	13.7	9.6	24.9	0.68	0.57
			11,000	22.1			9.5	6.5		0.77	0.58
Based on Actual Use											
Penobscot, Maine.....	1918-1922	1,910	41,600	21.7			9.3		23.0	0.69	
Deerfield, Vermont—Somerset reservoir.....	1914-1922	30.4	2,700	89			38		32.4	0.57	
Deerfield, Vermont—Harriman reservoir.....	1924-1929	154 (a)	5,000	27.5			11.8		30	0.80	

(a) Below Somerset reservoir.

A high degree of storage development will be noted for La Loutre, the Aziscohos, Somerset, Lake Almanor, and San Joaquin (ultimate) reservoirs, and at Boulder Dam. The Matthis-Tallulah reservoir supplements the Burton, so that these two should be considered together as in 7 and 8. Lake Almanor, holding 171 per cent of the yearly run-off, is the most completely developed. Generally speaking, a capacity of from one-half to one-third of the mean yearly run-off is commonly utilized, depending upon cost and the head through which stored water can be used.

Use of Storage.—The proportion of the capacity of a storage reservoir used during the average year will vary with the method of operation, whether this is so conducted as to utilize the reservoir capacity each year as far as practicable or on the safe-yield basis to insure a certain dependable flow.

In Table 46 is given the proportion of the reservoir capacity that would be used during the average year for several proposed reservoir systems and from actual use of two reservoirs.

The studies upon which the first four sets of data in Table 46 are based were, with the exception of the Hudson River system, from investigations made by the author in the form of hydrographs, to determine the effect of proposed reservoir systems. From the results of the hydrograph studies flow-duration curves showing the effect of storage upon flow were prepared. These flow-duration curves for the Kennebec River are shown in Figs. 39 and 40, pages 146–147, those for the Saco River in Fig. 49, page 174. The Hudson River flow-duration curves were as given in the report of the New York Water Power Commission.¹

The storage used in the average year was determined from the area between the curve of flow without storage and the curve as modified by storage regulation.

Referring to Table 46 it will be seen that with yearly use of storage the proportion used is from about 0.68 to 0.87 of the reservoir capacity, this ratio varying approximately inversely as the degree of storage in millions of cubic feet per square mile. A plot from the data in Table 46 supplemented by further data for various streams in New England shows the following:

Storage above regulation point		Yearly use of storage, proportion of storage used in average year
Inches depth	Million cubic feet per square mile	
2.1	5	1.20
4.3	10	1.00
6.5	15	0.85
8.7	20	0.75
10.7	25	0.65
12.9	30	0.60

¹ "Water Power and Storage Possibilities of the Hudson River," Plate VIII, 1923.

A relatively small storage reservoir may be filled and used more than once during a season, and, hence, the proportion of storage capacity used may exceed 1.00. As the degree of storage increases, this factor of use would become less and less, as the reservoir becomes so large that it would not fill in some years.

For the safe-yield method of storage operation, only about 0.50 to 0.60 of the storage capacity is used in the average year, reflecting, as already noted, the lesser amount of reservoir use by this method and tendency to waste water over the spillway.

The foregoing results are all from theoretical studies and do not take into account any losses of the stored water due to irregular load and insufficient pondage at power plants or waste due to leakage. Hence, depending upon the method of storage regulation, the actual use of

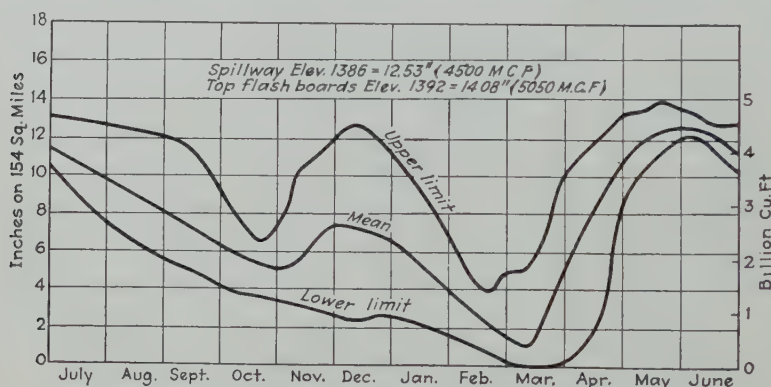


FIG. 42.—Davis Bridge (Harriman) Reservoir. Use of storage, 1924–1929.

stored water in the average year may vary from 50 per cent or less of the reservoir capacity up to nearly the full capacity under favorable conditions of use and with relatively small reservoir capacity.

Use of Storage on Deerfield and Penobscot Rivers.—For comparison with the preceding conclusions the actual use or release of storage at the Somerset reservoir on Deerfield River, Vermont, is of interest and is appended to Table 46. As previously noted (page 145) this reservoir has an unusually high degree of storage development, up to about 1.19 times the average yearly run-off, and the average use of storage capacity as shown by reservoir levels during the 9 years, 1914–1922, is about 0.57. During this period the reservoir filled (or substantially so) but twice, in June of 1916 and 1922. Its lowest monthly elevation was in September, 1914, when storage was about 95 per cent exhausted (due, however, in part to the reservoir filling only to about 85 per cent of its capacity the previous spring). The next lowest elevation reached was in February, 1918, when 90 per cent of the storage was exhausted. By years the capacity used during the year has varied from about 30 (1918) to 85

per cent (1916) of the full amount and as noted above averaged 57 per cent of the full capacity.

Since 1924 the Davis Bridge (Harriman) reservoir has also been in use on Deerfield River. The use of water for the period 1924-1929 is shown on Fig. 42, where the mean draft and upper and lower limiting curves of draft are shown. These indicate use of storage as follows:

Curve	Water stored, billion cubic feet			Proportion of storage capacity used yearly
	Maximum	Minimum	Range	
Mean.....	4.5	0.5	4.0	0.80
Upper limit.....	5.0	1.5	3.5	0.70
Lower limit.....	4.3	0.0	4.3	0.86

The Penobscot River storage is in Ripogenus reservoir (see Table 45), 25 bill. cu. ft., and Twin Lakes, 16.6 bill. cu. ft. The use of stored water is at and near Millinocket, just below the Twin Lakes. For the 4 years, 1918-1922, for which records are available the yearly proportionate use of storage capacity (computed from the discharge at Millinocket as published in *Water Supply Papers* 471, 501, 521, 541, and 561, with and without storage effect) is respectively 0.33, 0.83, 0.44, 1.00, and 0.86 and averages 0.69, which is fairly consistent with the results of the foregoing studies.

In the case of neither the Deerfield nor Penobscot rivers are these the actual use of stored water at power plants, this information not being available. The results given represent use of storage capacity at the reservoir.

PONDAGE

Pondage may be defined as the holding back and releasing later of water at the dam of a water power development (1) to equalize daily or weekly fluctuations in river flow or (2) to permit irregular hourly use of water by the wheels to accord with fluctuations in load demand. Fluctuations in river flow may be (1) natural, due to rainfall or snow melting, and (2) artificial, due to pondage of water at other plants upstream.

There is some confusion in the use of the terms pondage and storage. The latter refers more properly to the use of relatively large reservoirs, often distinct from power developments, to equalize monthly, seasonal, or yearly fluctuations in river flow. From the point of view of capacity alone, evidently an amount which on a large river might suffice only for pondage would on a small stream be sufficient for storage purposes.

To relate pondage capacity and rate of flow conveniently and quickly, it should be noted that 1 acre-ft. of water (43,560 cu. ft.) will provide

a flow of approximately 1 sec.-ft. for 12 hr. (43,200 sec. in 12 hr.) or $\frac{1}{2}$ sec.-ft. for a day.

To illustrate, the Vernon plant of the New England Power Company, on the Connecticut River, is well supplied with pondage facilities, with a pond about 25 miles long and a draft of 7 ft. obtained by the use of flashboards. This corresponds to about 16,300 acre-ft. of pondage, equivalent to a daily flow of about 8200 sec.-ft., which for the 6300 sq. miles of tributary drainage area is approximately 1.3 sec.-ft. per square mile, as compared with a mean river flow of about 1.7 sec.-ft. per square mile and a present wheel capacity (under a head of about 35 ft.) of about 2.0 sec.-ft. per square mile. This is evidently sufficient pondage to equalize river and load fluctuations under practically all ordinary operating conditions without serious fluctuations in head and is an important and valuable feature of this development.

On the other hand, Plant 3 of the same company, on the Deerfield River at Shelburne Falls, has a pond of only about 38 acres area, with a draft of 5 ft. or 150 acre-ft. of pondage, corresponding to only 75 sec.-ft. flow for 24 hr. or 0.15 sec.-ft. per square mile for the 500 sq. miles of drainage area. The mean flow of the river is about 2.3 sec.-ft. per square mile and the wheel capacity about 3.3 sec.-ft. per square mile (head = 64 ft.). Evidently pondage at this plant alone is sufficient to handle only ordinary low-water conditions. In this case, however, the next plant above (No. 4), which is adjacent to No. 3, has a considerably larger pond (430 acre-ft. of pondage) which is of effect and value in the operation of Plant 3, giving a total effective pondage of 580 acre-ft. or 285 sec.-ft. days or 0.57 sec.-ft. per square mile—enough pondage for medium and low-water flows.

Pondage Factors.—The term pondage factor (sometimes called multiplier or concentration factor) is often used, particularly with industrial plants, to express the flow (or power) available during the working day in terms of the 24-hr. flow (or power). For illustration, assuming a constant river discharge into the pond, with 10-hr. daily use of water, 6 days in the week, if water could be ponded during the other 14 hr. of the day, leaving Sundays out of consideration, it would be possible to obtain a 10-hr. flow $\frac{6 \times 24}{60} = 2.4$ times the 24-hr. rate of flow. If Sunday flow could also be ponded and used during the 6 weekdays, this pondage factor would be $\frac{7 \times 24}{60} = 2.8$. To make such concentration of flow possible in the 10 hr. of the day would require a pondage capacity to hold $1\frac{1}{2}$ times the 24-hr. flow in the first case (to be filled every night and drawn during 10 hr. the next day) and $\frac{14 + 24}{24} = \frac{38}{24}$ times the 24-hr. flow in the second case. In the latter case ideally the 24-hr.

flow would be ponded on Sunday and one-sixth of this released each weekday.

Thus, assuming a steady flow of 100 sec.-ft. and 10-hr. use 6 days per week, with perfect pondage, the following would result:

Item	Daily pondage	Daily and Sunday pondage
Assumed 24-hr. rate of flow, second-feet.....	100	100
24-hr. total flow, acre-feet.....	200	200
Pondage required, acre-feet.....	$1\frac{1}{2}\frac{1}{24} \times 200 = 116.7$	$3\frac{3}{24} \times 200 = 316.7$
Amount used daily, acre-feet.....	116.7	$116.7 + 20\% = 150$
Same second-feet in 10 hr.....	$116.7 \times 1\frac{2}{10} = 140$	$150 \times 1\frac{2}{10} = 180$
Flow during 10-hr. without pondage, second-feet.....	100	100
Total 10-hr. flow, second-feet.....	240	280
Pondage factor.....	$240/100 = 2.4$	$280/100 = 2.8$

Some waste of water usually occurs in operating pondage and often some leakage through flashboards of the dam or in other ways. Charles T. Main¹ suggests an allowance of 10 per cent for such losses in estimating pondage factors, or, for illustration, a maximum possible pondage factor for 10-hr., 6-day use of $2.8 \times 0.90 = 2.52$.

As will be further explained, waste and leakage losses may also be allowed for in the utilization factor adopted in estimating probable power output.

Pondage to Equalize Flow.—It must be kept in mind that the river flow into the pond is likely to be quite irregular, particularly if affected by plants upstream operating on an irregular 24-hr. basis, or, worse still, with substantially a day load only, which may itself fluctuate considerably. In these cases considerable pondage capacity may be required to handle and equalize this irregularity of flow into the pond.

As an example of actual river fluctuation during the low-water season, take a week's record of daily flow of the Connecticut River at Sunderland, Mass., about 4 miles above the head of the pond created by the Holyoke dam about 24 miles below Sunderland, as given in the table shown on page 156.

The daily flow at Sunderland is affected by pondage at the Turners Falls hydroelectric plant about 10 miles up the Connecticut River, and somewhat by plants on the Deerfield River, which enters the Connecticut a little below Turners Falls.

¹ *Jour. New Eng. Water Works Assoc.*, vol. 21, p. 214.

TABLE 47.—CONNECTICUT RIVER AT SUNDERLAND, AUG. 10-16. INCLUSIVE, 1919
(*Water Supply Paper 501, p. 115*)

Day	Dis- charge, second- feet	Per cent of mean for week	Departure from mean flow, second-feet		Ratio to 2320	
			+	-	+	-
Sunday, 10.....	1,510	65		810		0.35
Monday, 11.....	1,780	77		540		0.23
Tuesday, 12.....	2,650	114	330		0.14	
Wednesday, 13.....	2,580	112	260		0.11	
Thursday, 14.....	2,650	114	330		0.14	
Friday, 15.....	2,870	124	550		0.24	
Saturday, 16.....	2,200	95		120		0.05
Total.....	16,240	...	1470	1470	0.63	0.63
Mean.....	2,320	100				

As will be noted from Table 47, daily mean flow varies from about 65 to 124 per cent of the weekly average, and to maintain a steady flow equal to the latter would require a pondage capacity of 0.63 of this average, or 2940 acre-ft. The Holyoke pond, just below Sunderland, has an area of about 2400 acres and carries 2 ft. of flashboard, and would evidently handle this fluctuation in river flow with a maximum draft of pond of about $2940 \div 4800 \times 2 = 1.2$ ft., provided a uniform load was being carried by the wheels. The latter would vary, however, which might require some further pondage capacity to prevent waste of water over the dam at times. However, as this is the low-water season, it is of less importance to obtain equalization of flow, as any irregularity can be handled in part anyway by varying wheel gates or units.

A study similar to the foregoing made for the Merrimack River at Lawrence for the same week shows a daily mean discharge of 1705 sec.-ft., varying from a minimum (on Sunday) of 135 sec.-ft. (8 per cent of mean) to 2271 sec.-ft. (on Friday) (134 per cent of mean) and requiring pondage to equalize this mean flow of 1958 sec.-ft. one day or 3916 acre-ft. These data of discharge correspond to actual use of water out of the Lawrence pond, with an area of about 640 acres and 3 ft. of flashboards, giving 1920 acre-ft. of pondage. Evidently in this case use of water by the wheels must have varied very considerably to avoid waste of water over the dam. Note further that in the case of both the Connecticut and the Merrimack rivers the pondage effect shown by the tables is essentially that of Sunday flow used during the 6 weekdays. There must have also been considerable daily fluctuations in pond level due to the load carried at both Holyoke and Lawrence, which is principally industrial.

Pondage to Meet Irregular Load Demand.—Pondage required to permit the irregular use of water during the different hours of the 24-hr. day to accord with load demand or load curve will evidently vary inversely with the load factor, a 100 per cent load factor theoretically requiring no pondage and a low load factor a considerable amount of pondage. An inspection and study of the load curve in any case will enable the necessary amount of pondage to be determined. It is desirable, however, to determine approximately about what pondage is necessary for different load factors in terms of the average daily load, which may be done by assuming a generalized type of load curve as follows:

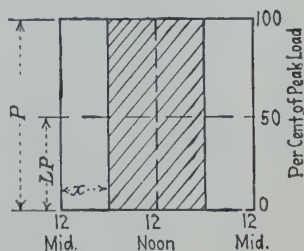


FIG. 43.

An assumption sometimes made is that as shown in Fig. 43 for a 50 per cent load factor, where the use of power is considered as concentrated into a continuous number of hours—in this case 12 hr.—consistent with the load factor. Thus if P = maximum or peak load, L = load factor:

$$LPx = (12 - x) (P - LP) = 12P - 12LP - Px + LPx$$

$$x = \frac{12P - 12LP}{P} = 12(1 - L)$$

where $2x$ is the time when plant is not running or the time of pondage. Pondage required will evidently be

$$2x \times \frac{LP}{24} = 24(1 - L) \frac{LP}{24} = (1 - L)LP$$

For several different load factors we should have:

Load factor	Corresponding hours of daily steady use	$2x$ = time of ponding, hours	Pondage required in terms of average load
1.00	24	0	0.00
0.80	19.2	4.8	0.20
0.60	14.4	9.6	0.40
0.50	12	12	0.50
0.40	9.6	14.4	0.60
0.30	7.2	16.8	0.70

While the foregoing is a simple and convenient assumption, it gives much too great a pondage for 24-hr. use of power, when compared with results obtained from the actual load curve. In Fig. 44 is given a typical load curve with 50 per cent load factor which requires a pondage of only about 0.20 the average load; the curve with 60 per cent load factor

(Fig. 44) requires a pondage of only about 13 per cent of the average load.

A better approximation of pondage requirements may be made by considering the 24-hr. day in four equal parts beginning with 12 mid-

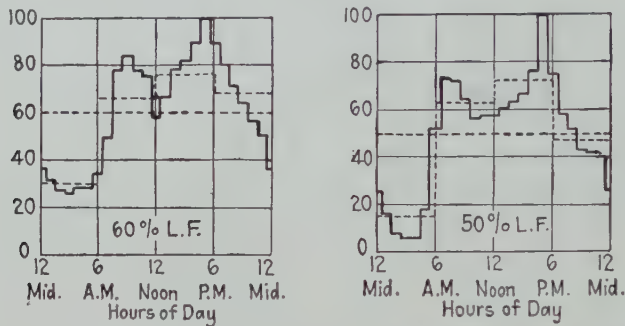


Fig. 44.—Typical load curves for 50 per cent and 60 per cent load factor.

night (as shown by the dotted lines on Fig. 44), and taking loads for these periods as follows for different load factors:

Load factor	Load in terms of peak				Approximate pondage required, proportion of mean daily load
	12 midnight–6 A.M.	6 A.M.–12 noon	12 noon–6 P.M.	6 P.M.–12 midnight	
1.00	1.00	1.00	1.00	1.00	0.00
0.80	0.60	0.80	1.00	0.80	0.07
0.60	0.29	0.68	0.80	0.63	0.13
0.50	0.15	0.64	0.73	0.48	0.20
0.40	0.00	0.50	0.60	0.50	0.25
0.30					0.33 ±

There is also given in this table, in the last column, the approximate pondage required to handle load fluctuations expressed in terms of the average daily load. As will be noted, these are only about one-half the amounts required by the previous table but are believed to be approximately correct for 24-hr. use and consistent with usual load curves. Note that these load curves for the high load factors indicate a fairly steady 24-hr. (industrial) load with moderate peak around 5 to 6 p.m. For the lower load factors the curves reflect essentially a day use of power, modified, however, by a considerable evening lighting load.

For working-day use of water—as for 10 or 8 hr., etc.—the pondage required as given in the first table would be approximately correct, using the column headed Corresponding Hours of Daily Steady Use rather

than load factor as an argument. The approximation in this case is that of assuming a steady working-day load. If, for illustration, the load at the beginning of the day was below the daily average, some additional pondage would be required to prevent waste of water at this time. Later in the day, with the pond somewhat drawn down, any load fluctuations could be carried by change in pond level.

Summary—Pondage Required.—1. Sunday pondage may require a capacity of pond equal to a day's average flow (or load) or for whatever period of time at the week-end shutdown may normally occur.

2. Daily pondage of night flow may require a capacity of pond of from about two-thirds of the average day load with an 8-hr. working day to no pondage with a 24-hr. working day.

3. Fluctuations in load, with 24-hr. use of power will require a pondage capacity of from about one-third of the average load for a 30 per cent load factor to no pondage with a 100 per cent load factor.

For the typical 24-hr. hydroelectric plant, even with a poor load factor of say 40 per cent, the pondage requirements to cover daily load fluctuations are thus seen to be only about one-fourth of the average load. Some lessening of Sunday load is likely, however, so that some pondage capacity must be reserved for this purpose. Fluctuations in river flow may also require considerable pondage to avoid very irregular use of water.

Pondage is, therefore, a very important factor in operating a hydroelectric plant, and must be carefully taken into account in planning any new development, particularly on a river where the flow is likely to be irregular due to pondage at other existing plants upstream.

THE USE OF STREAM-FLOW DATA FROM OTHER SITES

It is seldom that stream-flow records are available for a period of years at the exact site of a proposed water power development. Often a short-term record may be available, started perhaps as a part of the preliminary investigations for the project, but resort must usually be had to auxiliary records of flow: (1) at other points on the same river; (2) at points on neighboring streams of similar characteristics; (3) at points on streams not adjacent to that under consideration but nevertheless of similar flow characteristics; or (4) where no better information is available, estimation of run-off based on use of precipitation data for the drainage area tributary to the power site.

The use of any one of these auxiliary records of flow or of precipitation data, along with the short-time record at the power site, will be to establish for a long-time period the essentials usually required, *viz.*: (1) a flow-duration curve with or without storage effect, (2) the low-water flow, and (3) the flood flow to be expected.

Use of Auxiliary Records of Flow on the Same River.—It is often possible to assume without serious error that the unit flow (or that expressed in second-feet per square mile) of a stream is similar for its different subdivisions by tributaries or successive points along the stream. In such a case a flow-duration curve plotted on unit-flow basis may be used for any site on the river, the flow in second-feet being computed for the drainage area at the site under consideration.

It is usually necessary, however, to verify in some manner the use of unit flows at an auxiliary station. Where records are available at several stations in the drainage area this verification may be made by direct comparison of unit flows, either by calendar months or weeks or by a comparison of flow-duration curves. This should also be supplemented by a comparison of precipitation and, as far as practicable, other conditions affecting run-off, such as elevation, slope, soil, and cultural conditions.

If only one other auxiliary station is available, the use of its unit-flow data should be supported in all cases by comparative studies of the precipitation and other conditions just outlined.

For the comparison of unit flows at different points on the same stream or of different streams, a plot will be found useful, such as the unit flow of one point against the other, as illustrated in Fig. 45, where are shown for comparison the relative unit discharge for each month of the year at two stations on the Raquette River in New York for the period 1909–1913, inclusive. As will be noted, certain months other than those of winter and spring show fairly consistent relations, with a slightly greater unit discharge at Piercefield, the upper station. During the winter and spring the flow relations are more variable and discordant. In addition to showing relative unit flow, such a plot is useful in interpolating missing records of flow for either station but must be used with caution and results checked as far as possible from precipitation and temperature records.

Another useful method of graphical comparison is to plot for each month or period of time used a summation of flows at one station against the other, using the average slope of this resulting mass curve to define the relative station flows.

Where comparative studies—tabular or graphical—show that unit flows at the auxiliary station cannot be used for the power site without some correction, this should be made as consistently as possible from the results of the comparisons. It is often difficult to make such corrections with accuracy to monthly data of discharge. Yearly comparisons, as well as comparisons for the general subdivisions of the water-year (storage, growing, and replenishing periods) are usually more consistent in results and may be utilized to check or adjust monthly values.

Another method of treatment is to use stream-flow data for the auxiliary station. For illustration, in making up a flow-duration curve for the power site, ascertain the probable site-correction factor expressed as a coefficient which may be stated or applied in reaching the final conclusion.

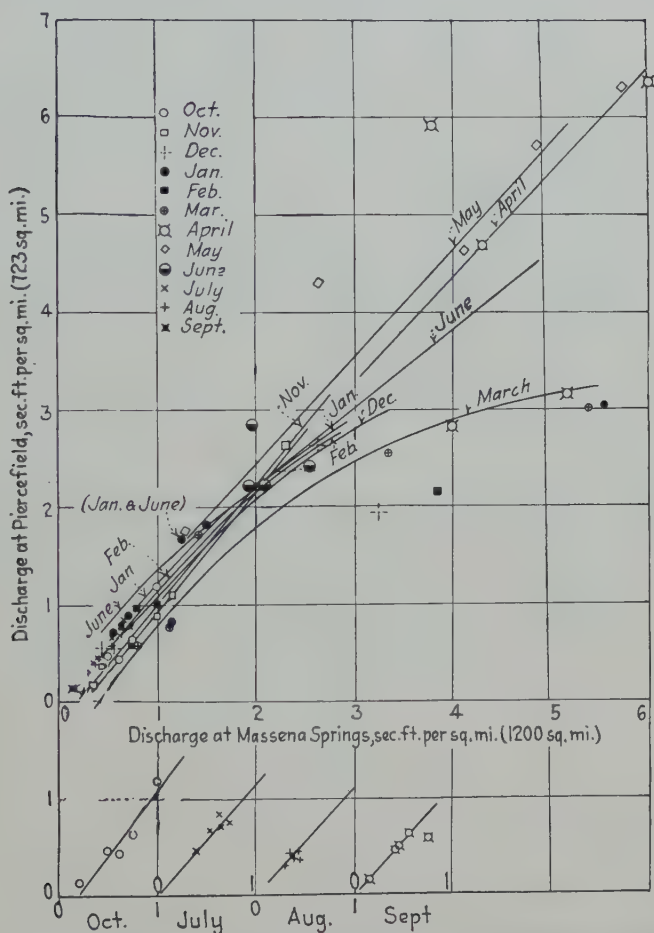


FIG. 45.—Comparison of mean monthly discharge of Raquette River at Piercefield and Massena Springs, N. Y.

Where the flow of a stream is affected by storage, care must be taken to correct or allow for this in using unit run-off data at points on the river below storage sites.

Thus for the month of August, 1919, on the Deerfield River in Vermont and Massachusetts the following data were observed:

DISCHARGE AND RAINFALL, DEERFIELD RIVER, AUGUST, 1919

Point	Drainage area, square miles	Measured flow		From storage above, point, second-foot	Flow corrected for storage		Monthly rainfall	
		Second-foot	Second-feet per square mile		Second-foot	Second-feet per square mile	Point	Amount, inches
Somerset reservoir, Vermont.....	30	107	3.57	68	39	1.30	Somerset	4.23
Charlemont, Massachusetts.....	362	269	0.74	68	201	0.56	Shelburne Falls	3.57

To estimate the probable flow at Plant 3, Shelburne Falls, at a point about 8 miles below Charlemont, where the drainage area is 501 sq. miles, the correct method of procedure would be:

$$Q = 269 + (501 - 362) \times \frac{269 - 107}{332},$$

$$= 269 + 68 = 337.$$

Applying the measured unit flow at Charlemont of 0.74 sec.-ft. per square mile would give $0.74 \times 501 = 370$ sec.-ft., which is too large, as this unit flow includes the effect of water released from storage in Somerset reservoir.

The above table also shows the greater natural unit flow in the more elevated regions of the Deerfield River. The water losses, etc., from these two points would be:

Point	Approximate rainfall above point, inches	Run-off, inches	Water losses, inches
Somerset reservoir.....	4.2	1.47	2.7
Charlemont.....	4.0	0.64	3.4

The greater unit flow at the Somerset reservoir as compared with that at Charlemont is due to both greater rainfall and smaller water losses at and above the former point.

In estimating the probable flow at points between Charlemont and the Somerset reservoir, a reasonable assumption would be that the natural flow at any point of this intervening area would vary between the amounts of 39 and 201 sec.-ft. at Charlemont and Somerset reservoir,

respectively, approximately directly as the proportion of the drainage area between the point in question and the Somerset reservoir to that between Charlemont and the Somerset reservoir.

Thus, at Davis Bridge (182 sq. miles) the estimated flow for June, 1919, would be

$$Q = 107 + (201 - 39) \times \frac{(182 - 30)}{(362 - 30)} = 107 + 74 = 181$$

At Charlemont there would be

$$181 + \frac{182}{362} \times (201 - 39) = 181 + 88 = 269, \text{ which checks}$$

Use of Auxiliary Records of Flow on Other Streams.—The use of auxiliary records of flow on other streams, either adjacent to or at some distance from the power site, must be supported and verified by precipitation and other studies with even more care than in the case of auxiliary stations on the same river. The general methods of making comparisons will, however, be similar to those just explained for stations on the same stream.

The effect of drainage-area characteristics and the necessity for their consideration in comparing or using stream-flow data for different streams are clearly shown in Table 38, page 111, previously referred to in discussing water losses, which states the essential results of a study made in 1915 by the author in connection with the valuation of water power at Seneca Falls, N. Y., to ascertain whether the Owasco River records (an adjacent stream) could properly be used as a basis for checking up the measured flow of the Seneca River. In this case so much storage capacity was available in Seneca Lake that the average yearly flow was a good index of available power and was used in the comparisons.

The following is taken from this table:

River	Precipitation, inches	Measured run-off, inches		Water losses	
		Land and water	Land	Land and water	Land
Seneca.....	34.57	13.5	15.0	21.1	19.6
Owasco.....	37.90	20.4	21.3	17.5	16.6

Using Seneca precipitation and Owasco water losses to compute Seneca run-off gives the following:

Seneca land and water run-off.....	17.1
Seneca land run-off.....	18.0

whereas the measured amounts for these two items are 13.5 and 15.0, respectively, differences of 3.6 and 3.0, due chiefly to difference in drain-

age-area characteristics as described in detail in Table 38, which result in a greater unit run-off for the Owasco River. Hence, even allowing or correcting for difference in precipitation is not enough to permit the use, without further correction, of Owasco run-off in estimating that of the Seneca River, even though the rivers be closely adjacent.

Use of Precipitation Data in Estimating Available Flow.—It will seldom be true, except in a new or unsettled country or region, that estimates of available flow must be based on precipitation data alone. The limitations in accuracy of such estimates will be obvious and much must be left to the experience and judgment of the engineer, who should be especially conservative as to his conclusions in such a case.

A study of the amount and distribution of precipitation and examination of the drainage area may even here make it possible to use as a basis for approximate water power estimates stream-flow data or a flow-duration curve for some river in a distant locality or even in another country.

A project of importance under such conditions would warrant obtaining records of stream flow for as long a period as may be consistent with the urgency for development. On the other hand, where a limited amount of water power is required, an examination of the stream and of precipitation records may be sufficient to establish the certainty of obtaining this power.

IMPORTANCE OF CORRECT ESTIMATE OF FLOW AND POWER

In concluding the discussion of this subject, the importance of correct estimates of flow and, hence, of available power is again emphasized. Many water power projects have been carried out with too little attention to this vital feature of successful operation, with consequent later embarrassment in operation. No matter how well a plant may be designed as to layout and equipment, if its water supply has been overestimated, the results may be financially disastrous. On the other hand, unnecessarily conservative estimates of flow may prevent a really good project from being developed.

While many excellent stream-flow data are being obtained and published by the U. S. Geological Survey in their *Water Supply Papers*, the importance of their work is not generally realized and has been hampered for lack of adequate appropriations. True and intelligent interest in water power development and the conservation of our fuel resources should stimulate a greater activity in this fundamental requirement for the proper estimation of available water power.

ESTIMATES OF AVAILABLE WATER POWER

Available Head.—The available gross head at a proposed power development is the fall from pond or reservoir level at the dam to water level

in the river where the tailrace is to be located. This fall or head will not always remain constant as the river discharge varies, because the rise in level of pond and that of the river at the tailrace are often different. Not infrequently the river level at the tailrace rises more rapidly than that in the pond above the dam, as discharge increases, resulting in "backwater effect" or less available head for higher stages of the river.

Backwater Effect.—The water level in the pond above a proposed dam can be readily computed for any given river discharge (making due allowance for water to be used by the wheels), provided the elevation and length of dam crest are fixed and the cross section of dam is known with sufficient accuracy to select a value of C in the weir formula $Q = CLH^{3/2}$.

These data in regard to the dam are usually fixed for a project, keeping in mind allowable limits in flowage upstream from the dam, if any, due to railroads, highways, etc., or sometimes to the tail-water level of an existing plant, which must not be exceeded.

The probable tailrace level at different river stages may be determined by installing a gage at this point and rating it by current meter measurements at different stages. By plotting curves of river elevation, (1) above the dam, and (2) at tailrace site, to the same datum, the gross head available at any river stage may be determined.

For low-head installations, say 50 ft. or less, especially on large rivers, loss of head due to backwater during the high-water season may be a matter of some consequence. As further explained in Chap. IV, any considerable decrease from the normal of the head acting on the wheels of an hydroelectric plant, which must run at substantially constant r.p.m., results in the wheel running at a speed which is not that for best efficiency, so that the power is lessened not only on account of head reduction but also by lower wheel efficiency. In planning a new development, therefore, possible backwater in the tailrace should be kept in mind and its effect studied, if necessary. Possible backwater effect at plants or privileges upstream, as a result of new dam construction, must also be considered.¹

Draft from Pondage.—In studying the effect of pondage, the decrease of head due to draft of pond must be allowed for. This may ordinarily be done by using the mean daily height of pond, which again will be approximately the mean of its maximum and minimum heights. If necessary, however, a more accurate graphical study of flow, load curve, and height of pond may be made to give the true weighted average of the latter during the day.

Often flashboards will be put on the dam for use especially in giving pondage during the low-water season, without material loss of head by draft from pond. In any event the tailrace level is at a minimum at

¹ See RUSSELL, "Hydraulics," pp. 252-255, 1925.

such times, so that frequently the normal head may be fully maintained, even with some draft from the pond.

Draft from Storage.—Where the dam is used for both storage and power development very considerable fluctuations in headwater level will occur, and the average head on the wheels must be determined from hydrograph studies of use of water from the reservoir, as previously explained. The reservoir-depletion curve in these studies will give a fairly accurate basis for the determination of the average available head.

The Davis Bridge reservoir and power development of the New England Power Company on the Deerfield River, Vermont (see page 152), illustrates this phase of operation. The reservoir may vary in level from about an elevation of 1392 to 1300 (see Fig. 42), while the tailrace level is at about an elevation of 998. The gross head, therefore, varies from about 390 to 300 ft. and averages perhaps 365 ft.

Net Available Head.—From the gross available head must be subtracted all losses between pond level and wheels, including rack and entrance losses to canal or penstock, frictional head, losses at bends in penstock, if any, etc. These may be accurately estimated after details of waterway, etc., are fixed but must also be allowed for in preliminary estimates of available power. As further explained under Waterways, the allowable loss of head is fundamentally that which will give a minimum annual cost of waterway plus value of power lost. It will usually be from 2 to 5 per cent of the gross head, depending upon the length and cost of waterway and value of power, and may be assumed as follows in preliminary studies:

TABLE 48.—PRELIMINARY ALLOWANCE FOR LOSS OF HEAD IN HYDROELECTRIC PLANTS

Gross head, feet	Loss of head, feet	
	From	To
25	1	2
50	1.5	3
100	2	5
200	4	8
500	10	20
1000	20	40

Available Power. *Limitation of Wheel Capacity.*—With a flow-duration curve available for the power site and the net available head ascertained, the available power may be estimated for any given wheel capacity by planimetry or scaling the area under the flow-duration curve up to the line of wheel-discharge capacity, thus obtaining the

average yearly flow to be utilized, which together with the average net head will give a basis for computation of the available power, assuming all water used. This may be done, using suitable flow-duration curves, for (1) the average year, (2) a dry year, and (3) a wet year, and the results given:

1. In horse power at wheel shaft.
2. In kilowatts at switchboard.
3. In kilowatt-hours yearly at switchboard.

The proper wheel capacity must be determined by studies or a consideration of cost of different-sized plants and the resulting output of power and the value of this power, as further explained in Chap. VII.

For hydroelectric plants this will commonly be found to be a wheel capacity corresponding to the flow available from about 20 to 40 per cent of the time, depending upon load conditions, particularly load factor, and the relation of the plant to other plants or interconnected transmission lines.

Classes of Water Power.—Water power may be classed as:

1. *Primary or firm power*, which must be always available and dependable for carrying load or that corresponding to the minimum stream flow, with due consideration of the effects of pondage and load factor. Primary or firm capacity may also be defined as the portion of total installed capacity which can perform the same function on that part of the load curve to which it is assigned as could be performed by an alternative steam plant.

2. *Secondary or surplus power*, which is power other than primary generated at a plant. The term "surplus power" is also sometimes used to distinguish the excess power available particularly at night, when due to low or no-load conditions there is no use for the power.

The 24-hr. power available 100 per cent of the time is not necessarily the limit in primary-power capacity if any pondage exists at the plant, as by the use of pondage, where the load factor is less than 100 per cent, the minimum available daily power capacity may be increased—sometimes several times over that capacity corresponding to the 24-hr. flow available 100 per cent of the time.

Use of Flow-duration Curve—Example.—Figure 46 is an example of the use of flow-duration curves in estimating power output for the water power development of the Amoskeag Manufacturing Company at Manchester, N. H., on the Merrimack River as developed at present. The mean curve is on the total-period basis, for the period Oct. 1, 1879, to Sept. 30, 1918, and is based upon the curves as shown in Fig. 31, assuming the flow between Garvins Falls and Lawrence (within which distance Manchester is located) to vary in direct proportion to intervening drainage area. These drainage areas are:

	Square Miles
Lawrence.....	4452
Manchester.....	2840
Garvins Falls.....	2340

Hence, as will be noted, the curve for Manchester lies at substantially three-fourths the distance from the Lawrence to the Garvins Falls curve.

Similar curves are also shown for a very dry year (1910-1911) and a wet year (1907-1908).

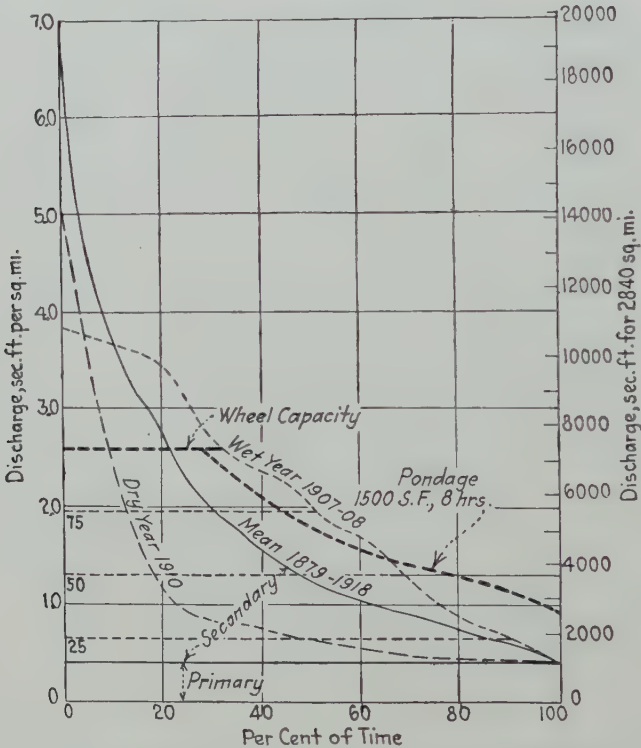


FIG. 46.—Flow-duration curves for Merrimack River at Manchester, N. H.

The use of the dotted line marked Pondage 1500 S. F., 8 hr., is explained on page 177.

The present wheel capacity at Manchester corresponds to a discharge of about 7400 sec.-ft. or the flow available about 22 per cent of the average year, there being wheels of about 31,000-hp. capacity, under a head averaging about 46 ft.

By means of planimeter or scaling, the following results may be obtained from Fig. 46, assuming at first no loss or waste of water, or 100 per cent utilization.

TABLE 49.—AVAILABLE POWER AT PLANT OF AMOSKEAG MANUFACTURING COMPANY,
MANCHESTER, N. H.
(From Fig. 46; 100 per cent utilization)

Curve	Average yearly output at wheel shaft, horse power (80 per cent efficiency)			Million kilowatt-hours yearly at switchboard (93 per cent efficiency)		
	Primary	Secondary	Total	Primary	Secondary	Total
Average year....	4800	12,800	17,600	29.0	78.0	107
Dry year.....	4800	6,000	10,800	29.0	37.0	66
Wet year.....	4800	16,700	21,500	29.0	102.0	131

Utilization and Capacity Factors.—Power output corresponding to the area below the flow-duration (or wheel-capacity) curve cannot be fully realized, due to:

1. A load factor less than 100 per cent, so that water power must be wasted at times.

2. In addition to wasting water over the dam at times, due to low load, there is likely to be some waste or non-use of water due to leakage, as through or under flashboards on the dam, through sluice gates, wheel pits, etc. With a plant in good order these losses should be small.

Load factor is defined as the percentage of the average load over a length of time (in this case, say, a year) to the maximum load over a short period of time, as a minute. Load factor is practically always less than 100 per cent. Where the load must be carried entirely by the wheels, it is evident that even with unlimited water supply an average output cannot be obtained in excess of that percentage of the wheel capacity which corresponds to the load factor, assuming that the plant is furnishing power up to its full capacity.

On the other hand the system load factor, where a plant is not an isolated one, may be different from the plant load factor, and it is often desirable to operate one or more plants of a system on a load factor higher than that of the system, carrying the irregularities of load at some one plant (sometimes a steam plant) adapted for this purpose.

Utilization factor is the ratio of water actually utilized for power to that available in the river, this latter being limited as shown by the flow-duration curve and (in the higher part of the curve) the discharge capacity of the wheels. With constant head, utilization factor would also be the ratio of power utilized to that available, and usually there would be little difference in this factor, whether expressed as a ratio of water or of power. As will be further explained, utilization factor commonly varies from about 0.40 to 0.90 for a hydroelectric plant, depending on plant capacity, load factor, available pondage and storage, etc.

Capacity factor or plant-use factor is the ratio of the average output of the plant to the plant capacity. It will be identical with load factor when the maximum load just equals the plant capacity, and these two factors are, therefore, often used as meaning the same thing. Capacity factor for a hydroelectric plant commonly varies from about 0.25 to 0.70 or more, depending on load factor, etc., as with utilization factor.

While utilization factor may approach 1.0 with a very high load factor and complete pondage at the plant, capacity factor would not exceed about 0.75 for the usual plant capacity, unless in addition to high

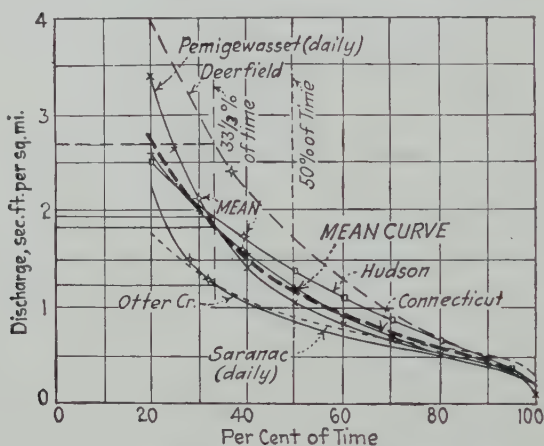


FIG. 47.—Flow-duration curves for various rivers.

load factor and complete pondage at the plant a large degree of regulation by storage is possible. A very low wheel capacity, for which water was available all the time, would also tend toward a high capacity factor. Thus, with complete storage, 100 per cent load factor, and complete pondage at the plant, and plant capacity equal to the average flow, the capacity factor would be 1.00 (with no waste or leakage). These conditions would, however, practically never be obtained.

Effect of Load Factor on Utilization and Capacity Factors.—To furnish a basis for general conclusion as to the effect of load factor, as well as form of flow-duration curve, upon utilization factor, Fig. 47 and Table 50 have been prepared, in which a considerable range in form of curve and amount of flow is covered. In Fig. 47 the assumed wheel capacity (also shown in Col. 4, Table 50) has been taken as corresponding to the flow available one-third of the time, which, as later shown in Chap. VII, is typical of the usual present development. Note also, as shown by Table 50, that this corresponds practically to the mean flow for the entire curve (or the mean yearly flow of the stream).

TABLE 50.—CHARACTERISTICS OF FLOW-DURATION CURVES

River (1)	Station (2)	Mean discharge, second-foot square mile (3)	Wheel capacity = flow available 33⅓ per cent of time				Per cent of time mean flow (Col. 3) available (8)	Ratio Col. 4 Col. 3 (9)
			Wheel capacity, second-foot per square mile (4)	Average discharge, second-foot per square mile (5)	Per cent of time Col. 5 available (6)	Ratio Col. 5 Col. 4 (7)		
Pemigewasset.....	Plymouth, N. H.	2.11	1.90	1.15	47	0.61	30	0.90
Otter Creek.....	Middlebury, Vt.	1.49	1.20	0.85	50	0.71	34	0.81
Deerfield.....	Charlemont, Mass.	2.36	2.66	1.66	50	0.62	37	1.13
Connecticut.....	Sunderland, Mass.	1.65	1.82	1.18	51	0.65	38	1.10
Saranac.....	Plattsburgh, N. Y.	1.27	1.25	0.87	51	0.70	32	0.98
Hudson.....	Mechanicsville, N. Y.	1.71	1.93	1.31	53	0.68	39	1.13
Mean.....		1.76	1.79	1.17	50	0.68	35	1.01

The average discharge (Col. 5, Table 50) corresponds to the area under the flow-duration curve and wheel-capacity line, and, as will be noted (Col. 6), this is closely the flow available 50 per cent of the time. Plant capacity factor with 100 per cent load factor is shown in Col. 7, averaging approximately two-thirds. It must be kept in mind that none of the curves shown in Fig. 47 reflect the use of any considerable amount of storage. If such were available and utilized, it would raise the right-hand portion of the flow-duration curve and increase the utilization factor for the higher load factors.

TABLE 51.—UTILIZATION AND CAPACITY FACTORS—TYPICAL FLOW-DURATION CURVES
(Plant capacity = flow available one-third of time)

River	Plant utilization factor				Plant capacity factor			
	System load factor, per cent				System load factor, per cent			
	100	75	50	25	100	75	50	25
Pemigewasset.....	1.00	0.87	0.68	0.40	0.61	0.53	0.41	0.24
Otter Creek.....	1.00	0.86	0.66	0.36	0.71	0.61	0.46	0.26
Deerfield.....	1.00	0.85	0.65	0.37	0.62	0.53	0.40	0.23
Connecticut.....	1.00	0.85	0.65	0.37	0.65	0.55	0.42	0.21
Saranac.....	1.00	0.75	0.55	0.28	0.70	0.60	0.44	0.23
Hudson.....	1.00	0.85	0.64	0.35	0.68	0.58	0.43	0.24
Mean.....	1.00	0.84	0.64	0.35	0.68	0.57	0.43	0.23

The available flow and corresponding utilization factors have also been worked out for 75, 50, and 25 per cent system load factors and shown in Table 51, assuming that (1) power will be taken up to the average assumed load (*i.e.*, wheel capacity times load factor) and (2) pondage is available sufficient for average load requirements and less, and that all water as shown by the flow-duration curve in amount less than the average use will be utilized without waste or loss.

This is consistent with a water power plant with supplementary steam power, with maximum yearly load just equal to wheel capacity, average yearly load equaling the proportion of wheel capacity consistent with the system load factor and all water used when available for power up to the amount of the average yearly load. It is to be further noted that, as assumed, plant load factor and capacity factor are identical but are not the same as the system load factor (system here meaning the water power and steam plant).

As will be noted, there is no great divergence of results in this table, and the mean values show for the various assumed load factors what

may be expected for utilization and capacity factors under the assumed operating conditions.

Effect of Plant Capacity on Utilization and Capacity Factors.—The foregoing results are based upon the usual plant capacity equivalent to the flow available one-third of the time. To show the effect of different assumptions as to plant capacity, using a mean flow-duration curve based upon the six curves in Fig. 47, utilization and capacity factors for plant capacities on a 25, 33 $\frac{1}{3}$, 40, and 50 per cent flow basis have been computed for various system load factors and the results plotted in Fig. 48, other assumptions and conditions as before.

The group of curves to the left of the 45-deg. dotted diagonal line in Fig. 48 gives utilization factor as ordinates, in terms of system load factor as abscissas, each curve representing some assumed wheel capacity

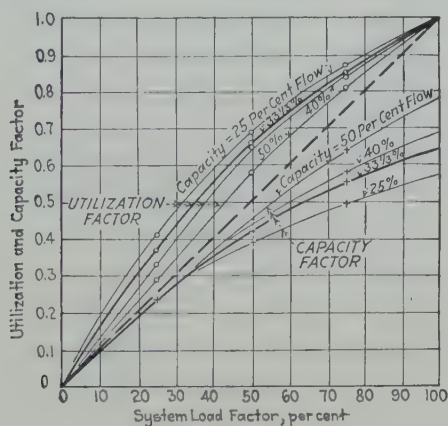


FIG. 48.

expressed as a percentage of time-flow basis. The curves at the right of the 45-deg. diagonal show capacity factor (and in this case plant load factor).

The 45-deg. dotted diagonal line is the limit for each of these two groups of curves. Then, with utilization factor, if system and plant load factors are the same, utilization factor becomes the same as load factor. With capacity factor, whereas the ordinary flow-duration curve will be of the general form shown in Fig. 47, if complete storage is available (the flow-duration curve then being a horizontal line), and system and plant load factors the same, then capacity factor becomes the same as load factor and again the limiting curve is the 45-deg. diagonal line.

With leakage and waste minimized, as with a well-operated plant, it is evident that utilization factor may vary between 0.30 and 1.00 while capacity factor will ordinarily range between about 0.3 and 0.7. Note

also from Fig. 48 that a larger wheel capacity tends toward a higher utilization factor but a lower capacity factor.

Effect of Partial Storage Regulation on Utilization and Capacity Factors.—The effect of partial storage regulation upon the flow-duration curve is shown in Fig. 49, where are shown for the Saco River at West Buxton, Me.¹ (1) the curve without storage or the natural river flow upon a drainage area of 1550 sq. miles for the period 1908–1916 by months; (2) the curve as modified by storage of 14 bill. cu. ft., assuming this curve as a mean of that for best primary power and that for yearly use of storage.

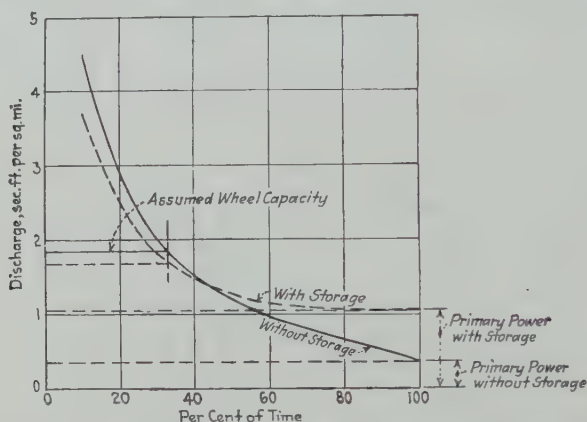


FIG. 49.—Saco River at West Buxton, Me. Effect of storage regulation on flow-duration curve.

The curve with storage is higher at the right of the 43 per cent line and lower at the left of this line than the curve of natural flow. Note also that the assumed wheel capacity for the flow available one-third of the time is about 1.67 with storage and 1.83 without storage, a slight lessening with storage as already noted.

The form of the curve without storage is almost identical with the mean curve in Fig. 47, as will be noted from the values of utilization and capacity factors, without storage, in Table 52.

TABLE 52.—EFFECT OF STORAGE REGULATION ON UTILIZATION AND CAPACITY FACTORS

System load factor, per cent	Plant utilization factor		Plant capacity factor	
	Without storage	With storage	Without storage	With storage
100	1.00	1.00	0.66	0.80
75	0.86	0.87	0.57	0.70
50	0.67	0.62	0.44	0.50
25	0.37	0.32	0.25	0.25

¹ *Jour. Boston Soc. Civil Eng.*, January, 1921, p. 29, Fig. 8.

There is little change in utilization factor with this degree of storage except at the lower load factors, where it becomes somewhat less, indicating the desirability of a relatively high load factor where storage has been made available. Capacity factor, with storage, is materially higher for all except low load factors. Here again the desirability of high load factor to utilize best the results of partial storage is obvious. Note also that with storage the primary power is about 80 per cent of the output with 100 per cent load factor, while, without storage, primary power is only about 30 per cent of the output, reflecting the great gain in primary power due to storage, provided a high load factor is maintained.

Effect of Lack of Pondage on Utilization and Capacity Factors.—In the foregoing discussion it has been assumed that pondage is available to the extent necessary to take care of the average load, its fluctuations varying with the load factor. As already noted (page 158), pondage requirements to cover daily load fluctuations will not be over an amount of water corresponding to about one-third the average load for 24 hr., even with a low load factor. If Sunday load is low, however, an additional amount may be required reaching a day's average flow if the plant is wholly shut down on Sundays.

With limited pondage a study must be made to determine a corrected flow-duration curve that will take into account loss of water due to lack of pondage, for the particular load factor expected.

Utilization and Capacity Factors as Obtained.—The various elements affecting utilization and capacity factors have been discussed and their ideal value estimated within limits. Actual values would be expected to depart somewhat from those suggested, as the studies have been made using an average of ideal flow-duration curve, whereas the actual daily flow may be fluctuating through wide limits during relatively short periods. There is, moreover, always a greater or less actual loss of water due to leakage through flashboards, bulkhead walls, gates, wheel pits, etc.

In Table 53 are given monthly data of output, utilization factor, and capacity factor for the years July 1, 1924, to June 30, 1928, inclusive, for the six Deerfield River plants of the New England Power Company and the Vernon plant of this company on the Connecticut River; also for the calendar year 1924 for the Indian Rapids plant on Saranac River at Plattsburgh, N. Y.

For the Deerfield River plants the available flow was estimated from (1) the flow from the Somerset reservoir (30 sq. miles) (above the Searsburg plant 98 sq. miles); (2) the flow from Davis Bridge reservoir (182 sq. miles) determined from power output, as during this time no water was wasted over the spillway; (3) the flow at Charlemont (362 sq. miles), a U. S. Geological Survey gaging station, between Plants 5 and 4.

TABLE 53.—UTILIZATION AND CAPACITY FACTORS (U.F. AND C.F.)

Plant	Searsburg	Harriman (formerly Davis Bridge)	No. 5	No. 4	No. 3	No. 2	Vernon	Indian Rapids	
Conditions									
Kilowatt capacity.....	5,000	45,000	17,000	6,000	6,000	6,000	25,000	600	
Head, feet.....	230	360	240	64	66	60	33	14	
Pondage, second-foot days.....	81	56,000	100	215	70	201	8,200	Very small	
System load factor.....					New England Power Company = 42 per cent about				
								About 55 per cent	
Month	Output, million kilowatt-hours	U.F.	C.F.	Output, million kilowatt-hours	U.F.	C.F.	Output, million kilowatt-hours	U.F.	C.F.
Jan.....	2.6	0.85	0.72	17.9	0.64	0.46	6.0	0.87	0.45
Feb.....	2.0	0.87	0.61	11.4	0.56	0.34	5.7	0.97	0.32
Mar.....	2.7	0.89	0.74	6.5	0.29	0.18	4.0	0.89	0.27
Apr.....	2.9	0.89	0.85	4.0	0.18	0.09	2.4	0.92	0.52
May.....	2.4	0.92	0.66	4.7	0.21	0.16	1.6	0.87	0.63
June.....	1.6	0.87	0.45	8.8	0.41	0.24	1.8	1.00	0.50
July.....	1.8	1.00	0.50	9.3	0.46	0.24	1.9	0.97	0.43
Aug.....	1.9	0.97	0.54	8.9	0.34	0.19	1.6	0.94	0.48
Sept.....	1.6	0.94	0.47	7.9	0.34	0.19	1.9	0.82	0.57
Oct.....	1.9	0.82	0.53	7.1	0.32	0.19	2.3	0.72	0.48
Nov.....	2.3	0.72	0.66	5.7	0.26	0.15	2.6	0.86	0.50
Dec.....	2.6	0.86	0.73	10.3	0.58	0.33			
Year.....	26.3	0.88	0.62	102.5	0.37	0.24	26.3	0.88	0.46

For Vernon (6300 sq. miles) the flow of the Connecticut River at White River Junction (4120 sq. miles) and at Sunderland (8000 sq. miles) was used to estimate the available flow at the plant.

For Indian Rapids, discharge as measured by the U. S. Geological Survey was used, this being a regular station of the Survey.

Power output and other data were furnished through the courtesy of the New England Power Company and the computations made under the direction of the author as thesis work by R. W. Frost and others at Massachusetts Institute of Technology.

The Deerfield River stations all show high yearly utilization factors, varying from 0.84 at Plant 2, farthest downstream, to 1.00 at Davis Bridge, where large storage capacity is available. The relatively low yearly capacity factor of 0.37 at Davis Bridge is due to the fact that this is operated essentially as a day-load station, with large wheel capacity, approximating the flow available only 5 per cent of the time. The Searsburg plant also has the high yearly utilization factor of 0.88, due in part to the relatively large storage in Somerset reservoir above this plant, and also because this is an automatic station and run at a relatively high capacity factor.

The Vernon station also has a relatively high yearly utilization factor due to excellent pondage facilities and fairly good load factor.

The weighted yearly station capacity factor for the seven New England Power Company plants in Table 53 is 0.47, for a total output of 425 mill. kw.-hr., whereas the system load factor is about 0.42, with a total output of about 700 mill. kw.-hr. The other power of the system not included in Table 53 includes both water and steam power.

The Indian Rapids station shows a low yearly utilization factor due to very small pondage facilities and low load factor.

The seasonal variation in utilization factor for the different types of stations is also of interest in Table 53. In general this factor is higher in summer and fall, falling off during the winter and spring months. Data in Table 53 should be studied in comparison with Fig. 48. Further information regarding actual utilization and capacity factors is desirable for a wider range of plant conditions, for use in estimates of probable power output.

Effect of Pondage on Power Output.—Where pondage is available the power output may be more or less concentrated into the day hours, as previously explained. This will not increase the kilowatt-hours of output but will increase the primary horse-power capacity and enable a larger load to be carried in the day hours. Thus, in the case of Fig. 46, page 168, pondage is available at Manchester sufficient to retain about 1500 sec.-ft. for 8 hr. Hence, the day (8-hr.) use of power for the mean curve can be increased as shown, if desired, to the line marked "Pondage 1500 sec.-ft. —8 hr." and the average daytime output raised from about 17,600 to

22,300 hp. (with 100 per cent load factor). During the 16 other hr. of the day the power will be correspondingly decreased, however, the kilowatt-hours for the day of 24 hr. not changing. The primary 8-hr. horse-power capacity in this case, taking into account the effect of pondage, is about 11,800 hp. instead of 4800 hp. as in Table 49.

It is obvious that the amount of primary power at a hydroelectric plant should be carefully determined in each case, taking into account the minimum river flow, available pondage, and the probable load curve.

Head-discharge Curve.—Where there is material variation in head with change in river stage, it may be necessary to take account of this by a head-discharge curve plotted perhaps on the same diagram as the flow-duration curve so that the correct available head may be used for each part of the flow-duration curve. In the case of the example given for the Manchester plant, on page 168, there is no great variation in head and no error will result to assume it constant.

Peak-load Hydroelectric Plants.—If the hydroelectric plant is in a power system and is to be used to carry the system peak, the conditions under which it must be run and the required pondage may be determined by plotting the system load curve hour by hour for the maximum day and determining by hour-to-hour study to what extent the hydroelectric plant can carry the peak, taking into account its available pondage.

For preliminary estimates Table 54 will be found useful in determining the approximate extent to which hydroelectric power can carry daily loads under different load factors and the extent of pondage required. This table was prepared by plotting "per cent of peak load" against "per cent of energy" for various different load curves, through the range of load factors shown. The column headed Hours of Use gives the approximate time during which power would be used, which subtracted from 24 hr. gives the approximate time during which pondage would be required.

Pumped Storage.—As will be noted from Table 54 the percentages of daily power output required to supply the upper portion of the load curve are relatively small. Thus the upper 25 per cent of capacity, under a 50 per cent load factor carries only 4 per cent of the total daily power output, and 50 per cent of the total capacity carries but 20 per cent of the daily output. Much of the plant capacity under these conditions is in use but a relatively small part of the time.

The use of auxiliary "pumped-storage" plants¹ has been suggested to carry peak loads, and several such plants are in use abroad and one, *viz.*, the Rocky River plant in this country. They require a reservoir near the main plant for pondage, preferably at sufficient elevation above it that water from the reservoir can be used to develop hydroelectric power

¹ FREEMAN, W. W. K., "Pumped Storage Hydroelectric Plants," *Trans. A.S.C.E.*, 1930, pp. 884-935; also *Power*, January, 1934.

under relatively high head and with, therefore, low-cost plant and equipment. Motor-operated pumps at the main plant, which may be steam or hydroelectric, pump water from river or other source of supply to reservoir at times of off-peak with either low-cost off-peak steam power or surplus hydroelectric power, and this is used to provide power at times of peak load. The economy of such a scheme lies in the great improvement in load factor at the main plant and consequent saving in equipment.

TABLE 54.—SYSTEM LOAD CURVES—APPROXIMATE PEAK AND OFF-PEAK-LOAD RELATIONS

Per cent of peak	70 per cent load factor			60 per cent load factor			50 per cent load factor		
	Hours of use	Per cent of daily energy		Hours of use	Per cent of daily energy		Hours of use	Per cent of daily energy	
		Below load curve	Above load curve		Below load curve	Above load curve		Below load curve	Above load curve
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
100	0	0.0		0	0.0		0	0.0	
95	3	1.0		1	0.7		1	0.5	
90	5	3.0		2	1.5		2	0.8	
85	6	5.0	25.0	3	2.5		2	1.5	
80	7	7.5	21.0	5	4.0		3	2.0	
75	8	11	18.0	6	6.0	30.0	5	3.5	
70	9	14	14.0	7	9.0	27.0	6	6.5	
65	10	18	11.0	9	13	23.0	7	8.0	35.0
60	11	22	8.0	11	18	18.0	8	11	30.0
55	13	27	5.0	13	22	15.0	9	15	25.0
50	17	32	3.0	13	27	12.0	10	20	20.0
45	18	38	2.0	14	33	9.0	12	26	16.0
40	20	44	1.0	15	40	6.5	15	32	12.0
30	24	56	0.0	18	50	3.0	17	44	6.0
20	24	70	0.0	24	66	0.0	21	60	1.0
10	24	86	0.0	24	80	0.0	24	80	0.0
0	24	100	0.0	24	100	0.0	24	100	0.0

The Rocky River plant (see Chap. XII, page 679) has an unusually large reservoir and makes use (as do some of the European plants) of seasonal storage of pumped water, as well as handling daily peak loads.

The Baldwin-Southwark Corporation (I. P. Morris Division) of Philadelphia, together with the General Electric Company, have recently developed a combined pump-turbine operated by a two-speed motor generator for such plants, model tests of which indicate relatively high efficiencies when acting as either a turbine or a pump.

RESERVOIR EFFECT ON FLOOD FLOW

In estimating the probable flood to be safely carried by the spillway and gates of a water power development, the presence of either natural

or artificial reservoirs upon the stream above the plant may considerably decrease the amount of water to be handled and particularly lower the amount of the crest or peak flow of the flood.

In such cases it is not desirable to count upon any storage of flood waters below full-reservoir level, because sooner or later the flood may occur at a time when the reservoirs are filled. The slowing down of discharge due to storing of water in the reservoir above its spillway level may, however, be sufficient to greatly lower the flood peak and in some cases, where large reservoir areas exist, even to decrease the average 24-hr. discharge during the day of greatest flow.

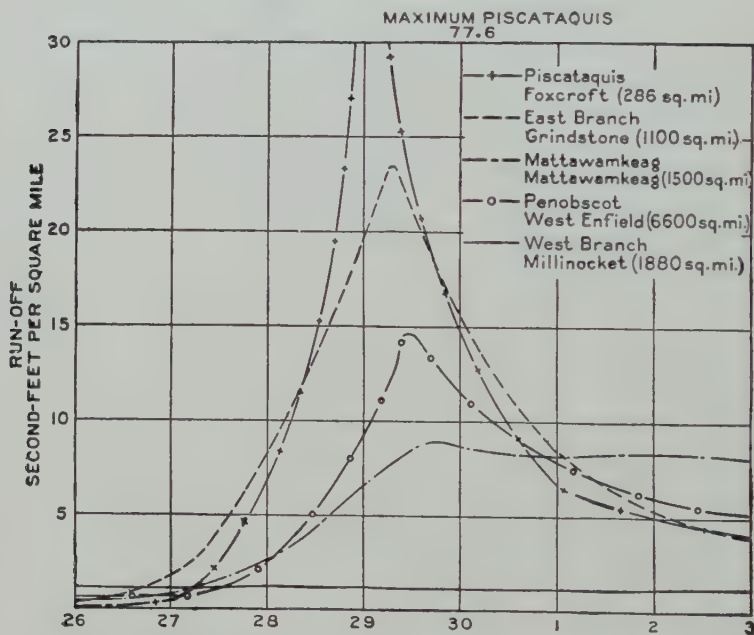


FIG. 50.—Penobscot River—flood hydrographs, September, 1909.

Methods for studying the effect of reservoirs upon flood flow will, therefore, now be considered.

Flood Curve or Hydrograph.—The flood hydrograph for a river without storage, and where conditions of basin slope, soil, etc., favor quick run-off, will have steep slopes, somewhat steeper during the time of increasing flow, and a relatively sharp peak. The curve will be rounded and the peak lowered when basin conditions favor large percolation rather than surface flow, as in sandy basins with moderate slopes. In general, also, the curve will have flatter slopes and a more rounded peak for large basins, as compared with small areas.

Illustrative Flood Hydrographs.—In Fig. 50 are shown hydrographs for several stations on the Penobscot River, Maine, during the flood of

September, 1909, the greatest observed at most points on the river during the past 25 years or so.¹ This flood was noteworthy in that it was caused by excessive rainfalls (mostly in 3 days) following a period of drought, as shown by the following table.

PENOBSCOT RIVER RAINFALL, SEPT. 26-30, 1909

Station	Rainfall, inches					
	26	27	28	29	30	Total
Millinocket.....	0.0	2.15	1.49	3.23	0.15	7.02
Orono.....	0.65	2.06	0.87	2.672	0.0	6.25

On the Piscataquis River above Foxcroft there is practically no storage, and its steep slopes and basin conditions favor quick run-off. As will be noted (Fig. 50), the flood hydrograph is steep, with sharp peak.

On the East Branch above Grand Lake (about 500 sq. miles) there are several large lakes controlled by dams. At the time these lakes were fully drawn down and the gates open, so that water was only temporarily held back. The flood effect, however, was probably largely from the 600 sq. miles of basin below Grand Lake, but the hydrograph shows much flatter slopes and a more rounded peak than the Piscataquis, the peak also coming several hours later.

The West Branch above Millinocket was entirely controlled by storage, with resulting steady, ordinary flow. The Mattawamkeag, a large tributary from the east, has some natural lake storage and is a stream of relatively slow run-off, as will be noted from its hydrograph.

The resultant hydrograph at West Enfield, well down the main river, shows relatively flat slopes (especially at falling stage) and rounded crest.

In Fig. 51 is shown a flood hydrograph for the Deerfield River at Charlemont, Mass., for July 8-10, 1915, the highest water in some 20 years, accompanied by data of rainfall and run-off for the first 10 days of July.² As will be noted, the crest discharge was 45,000 sec.-ft., the maximum 24-hr. discharge 20,500 sec.-ft., and the rainfall nearly 5 in. at Somerset in the two days, July 8-9. The unit discharge is for 332 sq. miles, or the area at Charlemont, excluding the 30 sq. miles of the Somerset reservoir, which was not filled at the time.

The hydrograph shows very steep slopes and a sharp peak, reflecting the quick-spilling character of the basin. The ratio of crest flow to average 24-hr. flow is about 1.96 for the maximum 24-hr. period, whereas

¹ U. S. Geol. Survey, *Water Supply Paper* 279, pp. 132-136.

² *Jour. Boston Soc. Civil Eng.*, January, 1920, pp. 34-35.

by Fuller's formula (page 120) this would be about 1.35, indicating, as has already been noted, the inaccuracy of this factor in his formula, which evidently depends upon basin conditions as well as size of area.

Form of Flood Hydrograph.—Where flood hydrographs of a stream are available, even for a flood less than the maximum, their form of curve may be used in studying the effect of greater floods. Where no flood hydrographs are available, a curve for the maximum flood day may be constructed based upon a computed average 24-hr. and a crest flow.

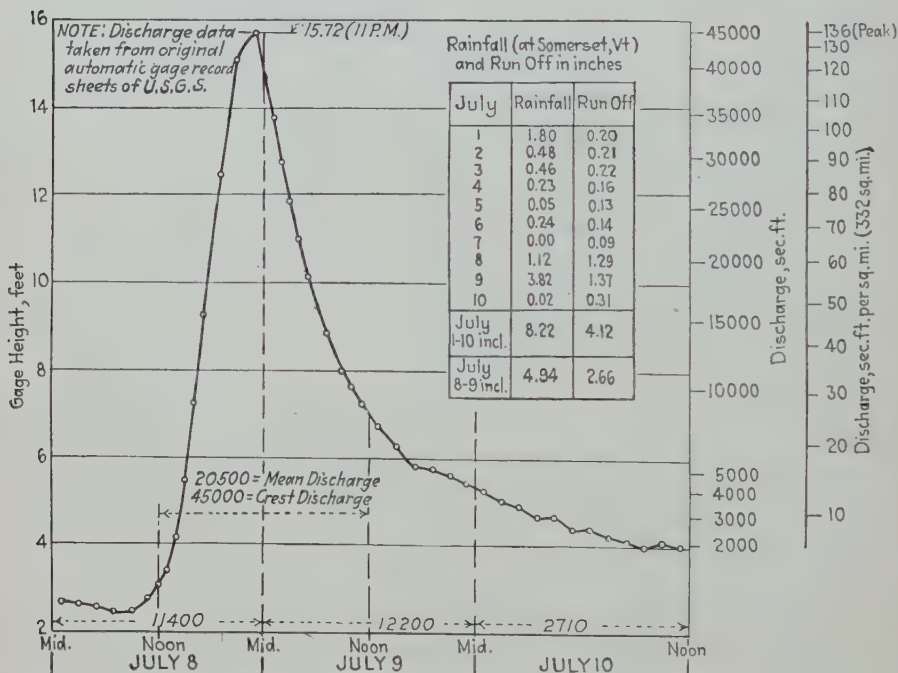


FIG. 51.—Deerfield River at Charlemont, Mass. Flood hydrograph, July 8-10, 1915.

For quick-spilling basins this curve may be taken as a straight line either side of the peak, or, in other words, the area formed by the hydrograph for the flood day will be a rectangle, surmounted by a triangle, the apex of the latter being assumed at noon of the flood day. For streams of slower run-off a parabolic curve may be used. The comparative forms of curve resulting from these two different assumptions are shown in Fig. 52.

Flood Hydrograph for Vermont Streams.—For the purpose of comparing flood characteristics of several streams in the same vicinity, the flood hydrograph may be plotted in terms of "per cent of peak discharge" instead of varying amounts of flow. This form of flood hydrograph was used by the author in a study of the Vermont flood of November, 1927,

as a basis for reservoir-spillway capacity.¹ Flood hydrographs of such rivers as were available were plotted and a mean curve determined which was adopted as a standard form of flood hydrograph for Vermont streams. This curve forms a triangle, starting at the beginning of the flood period with a flow equal to about 2 per cent of the peak flow, rising uniformly to 100 per cent at the twelfth hour and then decreasing uniformly to 20 per cent of the peak flow at the thirty-sixth hour. By using this curve for streams where the peak flow has been observed, an approximate flood hydrograph can be prepared for spillway-discharge studies. In the Vermont plans a peak discharge was assumed 25 per cent in excess of the 1927 observed peak.

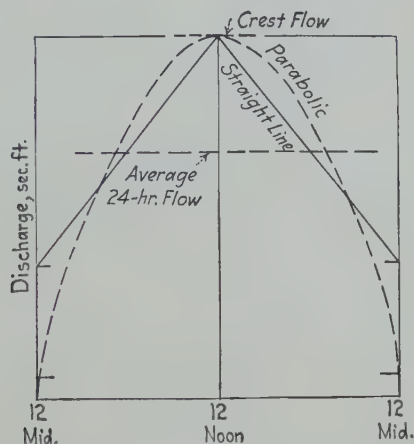


FIG. 52.—Forms of flood hydrograph.

Flood Flow as Modified by Storage.—With the flood hydrograph for the maximum day determined or assumed, the inflow to the reservoir or reservoirs may be obtained for each hour of that day, and the problem becomes one of determining the effect of water stored in the reservoir, as it rises above its normal level, in lessening the flow over the spillway of the reservoir dam.

Methods of Study.—Two different methods of treatment of this problem may be used: (1) a tabular time-flow increment method, and (2) a graphical mass-curve method.

1. **Time-flow Increment Method.**—This method consists of starting with water level at the height of spillway (or possibly at top of flashboards, if used) at the beginning of the flood day and making computations for each hour (or 2 hr. period) of the day, as shown by the following tabular headings:

¹ Report of consulting engineer to Advisory Committee of Engineers on Flood Control, Vermont, Fig. 2.

TIME-FLOW INCREMENT METHOD FOR SPILLWAY DISCHARGE—TABULAR HEADINGS

Hour (or period)	Flow into reservoir		Approximate rise of reservoir		Assumed mean head for hour, feet	Discharge over spillway		Net added to reservoir			Amount stored above spillway level, acre-feet
	Sec-ond-feet	Acre-feet for hour	For hour feet	Total feet		Sec-ond-feet	Acre-feet for hour	Acre-feet	Rise, feet	Eleva-tion of reser-voir	
1	2	3	4	5	6	7	8	9	10	11	12

The method of making these computations will be obvious, with the following suggestions:

Column 2: Flow into the reservoir should be the mean for the hour.

Column 4: The rise of the reservoir during the hour must be first approximated and the remaining computations (col. 5 to 10) made accordingly, this rise corrected if necessary, and the computations revised. As flow over the spillway is slight at first, it will be found easy to assume approximately correct values for col. 4 at once.

Column 7 is the flood discharge as modified by storage effect. In the solution it will be necessary to have an area curve or table for the reservoir at different elevations; also data of length of spillway and values of its discharge coefficient for different heads on the crest.

2. Mass-curve Method.—Numerous methods have been suggested, but the author has found the following, as devised by R. S. Holmgren of his office, most convenient.

On one diagram (Fig. 53) are plotted:

(1) A mass curve of reservoir inflow *A* prepared from the flood hydrograph, plotting time in hours as abscissas and total acre-feet of corresponding inflow as ordinates.

(2) Mass curves of flow over spillway *B* for any convenient period of time, as 10 hr., of a length to define accurately the slope of these straight-line curves with the same coordinates and scales as in (1). These curves are computed for rises of water level above spillway crest between heights of, say, 0 to 0.5, 0.5 to 1.0, 1.0 to 2.0 ft., etc., or intervals such that the average head on the spillway crest may be assumed without serious error to be the mean of the heights at the beginning and end of the interval.

(3) A curve of reservoir capacity above spillway level (or corresponding table of such capacities for the intervals assumed in (2) as shown on Fig. 53).

(4) Using the data in (2) and (3) a mass curve *C* of combined spillway flow and storage above spillway level is drawn starting at the zero point of curve *A*, the procedure being to first plot the storage capacity for interval 0 to 0.5 ft. as an ordinate, and from the top of this ordinate draw a line parallel to the mass curve of spillway flow for this interval in *B*. Where

this last line intersects curve *A* defines the time and total amount of water taken care of by the rise in level for the first interval (0 to 0.5 ft.).

At the point thus found is erected the ordinate of reservoir capacity for the next interval (0.5 to 1 ft.) and from the top of this ordinate a line drawn parallel to the mass curve of spillway flow for this interval in *B* to intersect curve *A* as before.

This process is repeated until a line parallel to some interval of height becomes tangent or approximately so to curve *A*. At this tangent point

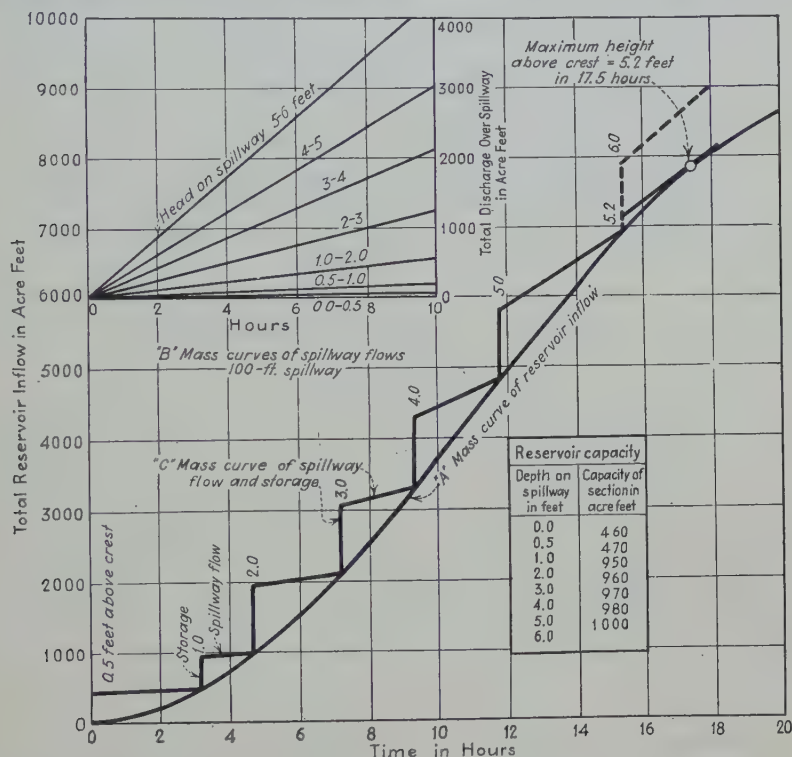


FIG. 53.—Mass curves of reservoir inflow and storage, and of spillway discharge.

and corresponding height of water over spillway reservoir, inflow is just equal to outflow over the spillway. Thereafter, outflow exceeds inflow and the reservoir water level will begin to fall. The maximum height of water above spillway level is thus obtained, as well as the time when this occurs, from the time scale of mass curves *A* and *C*.

The essential advantage of this method lies in the use of mass curves so as to secure a direct solution instead of the usual cut-and-try procedure. This direct solution is made possible by assuming intervals of successive heights on the spillway rather than successive intervals of time, as is commonly done.

Reservoirs for Flood Protection.—Sufficient storage capacity to equalize the flow of a river is seldom obtainable except upon relatively small streams. Referring to Table 45, page 149, in which are listed data of numerous power reservoirs, it will be seen that only three have sufficient capacity to approximately equalize the flow from their tributary drainage areas, *viz.*, the Somerset reservoir, holding about 1.19 times the average yearly run-off, Lake Almanor, with a proposed capacity about 1.71 times the yearly run-off, and La Loutre with 1.11 times the yearly run-off. On the other hand, storage capacity of one-third or more of the yearly run-off will tend materially to cut down flood peaks, although in such cases only storage above spillway level can be assumed as always available.

Storage capacity below spillway level in the case of power reservoirs may be of material effect in reducing flood tendencies—particularly in the northern and eastern sections of the United States or in other sections of the country where similar precipitation and run-off distribution occur. Normally in these districts power reservoirs, if effectively developed, are completely filled for only a month or two at the most in the late spring or early summer. Their level is gradually reduced during the year and the reservoir normally emptied, or nearly so, at the beginning of the spring run-off. Usually fall rains temporarily retard this draw-down to some extent and provide some added stored water.

Typical use of stored water from the Harriman (formerly Davis Bridge) reservoir of the New England Power Association is shown in Fig. 42 for the period 1924–1929. This reservoir with effective storage of about 42 mill. cu. ft. per square mile (including Somerset reservoir) or 0.57 of the mean yearly run-off (see Table 45, page 149) completely filled only once during these 5 years. It was very effective in preventing a flood on Deerfield River in November, 1927, at the time being drawn down to a level where it completely absorbed this flood, resulting in little or no damage on the river below, when on other Vermont streams, without storage, great damage with loss of life occurred.

Power reservoirs as normally operated may therefore provide relief from floods for a large part of the year. Even when a power reservoir is filled, if adequate gate capacity is available there is time after an unusual rainfall is under way to anticipate flood flow, by opening flood gates, lowering reservoir level, and thus providing storage in addition to that above spillway level.¹ Power reservoirs, properly operated and controlled, may therefore be effective in reducing flood tendencies even when filled.

Detention Reservoirs.—For flood protection alone, detention or retarding reservoirs may be constructed where conditions make this feasible. A detention reservoir is formed by a dam across a river valley, having in its base a conduit of such predetermined size that flood water

¹ See *Jour. Boston Soc., Civil Eng.*, September, 1930, pp. 430–431.

will be stored and redelivered to the stream at a rate dependent on the size of the conduit. By increasing the size of the reservoir basin and diminishing the size of outlet conduit, the flood flow may be lowered to the desired rate.

Detention reservoirs have been used in France dating back many years. In this country the five detention reservoirs constructed in the Miami River basin, Ohio, by the Miami Conservancy District are the first examples of the kind.

The hydraulic-fill dams for this work are described in Chap. V. The capacity and areas controlled by these reservoirs are as follows:¹

TABLE 55.—MIAMI CONSERVANCY DISTRICT—DETENTION RESERVOIRS

Reservoir	River	Drainage area, square miles	Reservoir capacity		
			Million cubic feet		Inches depth on drainage area
			Total	Per square mile	
Lockington.....	Loramie Creek	255	3,050	12.0	5.2
Taylorville.....	Miami	878 ^a	8,100	9.3	4.0
Englewood.....	Stillwater	651	13,600	20.9	9.1
Huffman.....	Mad	671	7,300	10.8	4.7
Total above Dayton.....	Miami	2500	32,050	12.8	5.5
Germantown.....	Twin Creek	270	4,630	17.2	7.4
Total above Hamilton.....	Miami	3500	36,680	10.5	4.5

(a) Not including area above Lockington reservoir.

The effect of such retarding reservoirs on flood flow is shown in Fig. 54, which shows an inflow like that of the March, 1913, Ohio flood to the Huffman basin and an outflow as it would occur through the conduits of this reservoir. The maximum inflow would be about 78,000 sec.-ft., the maximum outflow about 32,600 sec.-ft. (or about 42 per cent of the former), and storage capacity required for detention about 5.4 bill. cu. ft. or a depth of about 3.5 in. on the tributary drainage area, whereas the basin as constructed would hold about 4.7 in. depth.

The total storage capacity of the four detention reservoirs above Dayton—5.5 in. in depth on the 2500 sq. miles of drainage area—and of the five above Hamilton—4.5 in. in depth on 3500 sq. miles—as noted in Table 55, gives a fair idea of storage requirements of detention reser-

¹ Miami Conservancy District, *Tech. Reports*, Part VII, pp. 52-54.

voirs (or the portions of reservoirs thus used) for flood protection in the North and East, keeping in mind that some channel improvements were also made at Dayton and other points.

The maximum 3-day run-off above Dayton in March, 1913, was about 7.3 in. In the design of the retarding basins a run-off of 10 in. in 3 days and of $9\frac{1}{2}$ in. for the larger areas was assumed and conduits so designed as to handle this without overflowing the spillways. The total

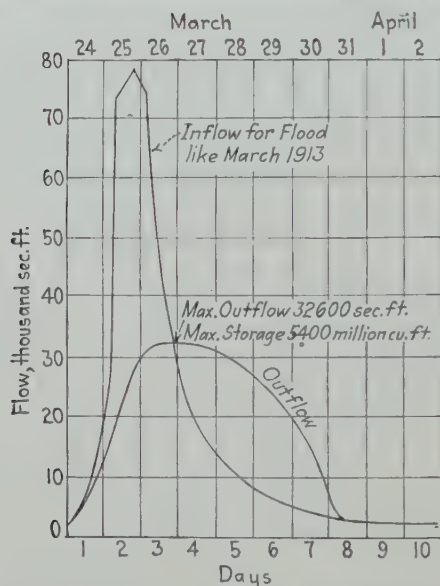


FIG. 54.—Huffman Detention Reservoir—Miami Conservancy District. Effect upon flood hydrograph.

retarding-basin capacity is, therefore, approximately half the assumed 3-day run-off.

The design of the outlet conduits in the Miami Conservancy dams is given in detail in Chap. VI of Part VII of their *Technical Reports*. An interesting feature of the design of these conduits is the use of the "hydraulic jump" at the outlet of the conduits, below the dam, to destroy velocity of flow without endangering the structure. This feature is discussed in Part III of these reports.

CHAPTER IV

HYDRAULIC TURBINES AND GENERAL ARRANGEMENT OF PLANT

NOTATION USED FOR HYDRAULIC TURBINES

- Q = flow or discharge in cubic feet per second or second-feet.
 w = weight per cubic foot of water, ordinarily 62.4 lb.
 W = discharge of water in pounds per second.
 H = total available head at water privilege.
 h = effective or net head on wheels.
 V = absolute velocity of water (relative to earth).
 v = relative velocity of water (relative to wheel runner).
 u = linear velocity of wheel runner for nominal diameter D .
 ω = angular velocity of wheel runner.
 p = pressure (expressed in gage units, as pounds per square inch).
 z = potential head (usually above tailrace level).
 1, 2, = subscripts for V , v , u , p , and z at entrance to and exit from wheel runner, respectively.
 h_u = head utilized by wheel.
 h_l = head lost in wheel, including all losses.
 h' = hydraulic losses.
 h_D = disk-friction losses.
 h_d = draft-tube losses.
 h_L = leakage losses.
 h_s = shock losses.
 D = nominal diameter of wheel runner in inches.
 N = speed of wheel in r.p.m.
 e = wheel efficiency expressed as a ratio of output to input.
 F = force applied to wheel runner or testing brake.
 T = torque applied to wheel runner or testing brake.
 ϕ = ratio $\frac{u}{\sqrt{2gh}}$ or relative speed.
 $h p_i$ = horse-power input to wheel.
 $h p$ = horse-power output from wheel (or b.hp.).
 ϕ_e = value of ϕ at full-load point (or point of best efficiency).
 C = discharge coefficient of a given wheel.
 $N_u = \frac{DN}{\sqrt{h}}$ = unit speed constant or speed in r.p.m. of a 1-in. wheel under 1-ft. head.
 $Q_u = \frac{Q}{D^2 \sqrt{h}}$ = unit discharge constant or discharge in second-feet of a 1-in. wheel under a 1-ft. head.
 $P_u = \frac{h p}{D^2 h^{3/2}}$ = unit power constant or power of a 1-in. wheel under a 1-ft. head.
 N_s = specific speed = speed in r.p.m. of a wheel delivering 1 hp. under 1-ft. head.

The hydraulic turbine as the prime mover of the water power development should be given some consideration and study before proceeding to an analysis of the different methods of water power development and the general features or elements which make up the plant. As will be seen, the type of turbine and manner of plant arrangement are largely interdependent, and it is, therefore, desirable to have first a clear conception of the turbine as a prime mover and its practical construction and types of setting. It is assumed that the reader has a knowledge of

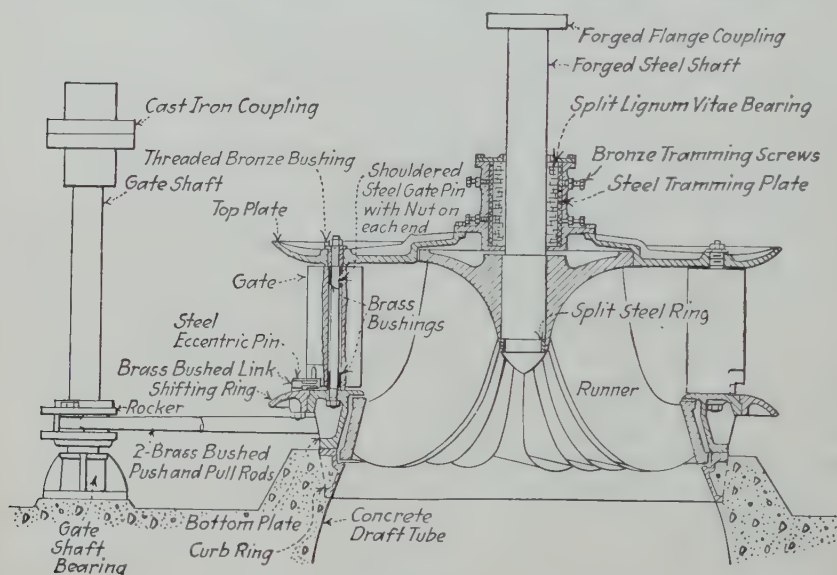


FIG. 55.—Reaction turbine, sectional view.

hydraulics, and the treatment will be of a nature to aid the engineer in the practical selection of turbines, rather than a discussion of theory.¹

Classification of Hydraulic Turbines.—The modern hydraulic turbine may be classified as (1) reaction and (2) impulse wheels.

In the reaction wheel, illustrated by Figs. 55 and 56, which show a sectional detail of a wheel and its general arrangement for an open wheel pit setting, the following conditions obtain:

1. The wheel passages are completely filled with water.
2. The water acting on the wheel vanes is under pressure greater than atmospheric.
3. The water enters all around the periphery of the wheel.
4. Energy in the form of both pressure and kinetic is utilized by the wheel.

The reaction wheel is the common type of water wheel as used today.

¹ For further study of the theory and basis for the design of hydraulic turbines the student is referred to Russell's *Hydraulics*, 1934.

In the impulse or tangential wheel (see Fig. 57), which shows the early simple arrangement of this type of wheel standardized and still in use for running installations or plants of a semipermanent nature:

1. The wheel passages are not completely filled.
2. The water acting on the wheel vanes is under atmospheric pressure.
3. The water is supplied at a few points at the periphery of the wheel (usually one point, but occasionally two or more points).
4. Energy applied to the wheel is wholly kinetic.

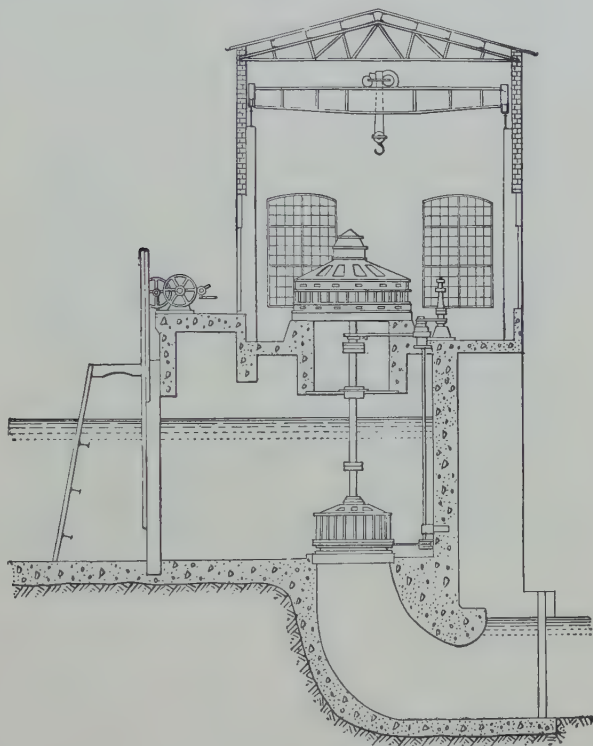


Fig. 56.—Reaction turbine, open flume setting.

The impulse wheel is used only for high-head developments and is the familiar bucket type of wheel actuated by a jet of water from a nozzle.

Reaction Wheels. *Direction of Flow.*—Reaction wheels may be classified with regard to the direction which water takes in flowing through them as follows:

1. Radial inward or Francis.
2. Radial outward or Boyden (or Fourneyron).
3. Mixed flow, radial inward and axial, or American.
4. Axial flow or Jonval (the recent modern developments of which are the so-called "propeller types" of runners).

Historical.—The first radially inward-flow wheel in the United States was built by Howd at Geneva, N. Y., in 1838, although the development of this type of wheel to a perfected state was by Francis in 1849 and it has always been known by his name.

The radially outward-flow or Fourneyron wheel was built by the French engineer of that name in 1827, followed in this country in 1844 and thereafter by many wheels of this type built by Boyden at Lowell, Mass. The Boyden wheel, as it was called, was in very general use in New England for many years.

Both the Francis and Boyden wheels gave good efficiencies—80 per cent or better in many cases. The Boyden wheel, however, was at a serious disadvantage structurally, the runner being on the outside of the wheel, thus requiring a larger and more expensive wheel for a given

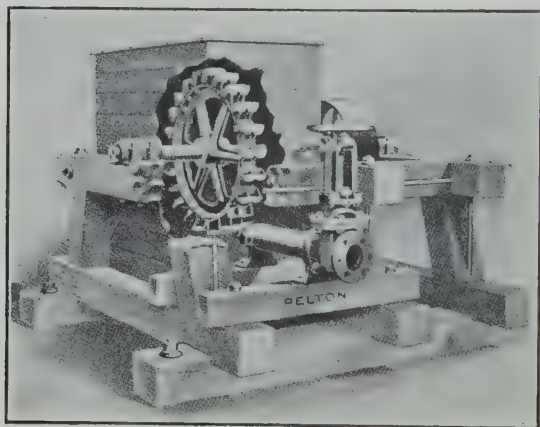


FIG. 57.—Simple form of impulse wheel.

amount of power and also operating at a much lower speed than the Francis wheel. In time this resulted in the complete abandonment of the Boyden wheel. An example of this type is the Kilburne-Lincoln wheel, made for many years at Fall River, Mass.

The last notable instances of outward-flow wheels installed in this country were during the period 1895–1900. At the Niagara Plant 1 of the Niagara Falls Power Company, 10 units with tailrace tunnel setting, each delivering 5000 hp., under 136 ft. head at 250 r.p.m. ($N_s = 38$) were built by the I. P. Morris Company from designs of Faesch and Picard of Geneva, Switzerland. At about the same time this American company constructed similar wheels from its own designs—four units, each delivering 1700 hp. under a head of 266 ft. at 360 r.p.m. ($N_s = 13.7$), for the Trenton Falls, N. Y., plant of the Utica Gas and Electric Company. These units, with the original bronze runners, are still in use and are of especial interest as being the first large high-head wheels designed and constructed in this country. The Niagara wheels at the No. 1 power

house previously referred to were replaced some time ago by units of the Francis type, which latter installation is now held as a standby plant, only, for the new No. 3C station recently completed, where three 70,000-hp. units are in use.

The axial flow or Jonval wheel was invented in 1837 by Henschel and introduced into this country about 1850. At this time the tubwheel, so called, was in use—an axial-flow wheel without guides, placed in the bottom of a flume. By fitting this with a cover containing guide pas-

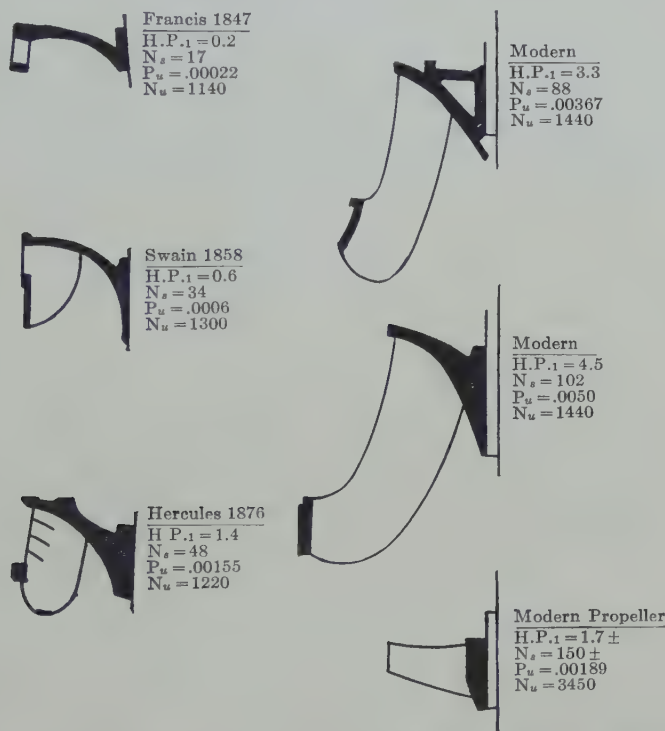


FIG. 58.—Evolution of American wheel. Bucket outlines of runners of approximately the same rated diameter. Horse power on basis of 30 in. diameter.

sages, the wheel became essentially a Jonval. The Jonval wheel was made at several locations in New York up to about 1900 (by McElwain, Dix, etc.) and used extensively in that part of the country. It is not now manufactured in this country, although a few old wheels are still in use.

Development of the Mixed-flow or American Type of Wheel.—The Francis wheel as originally constructed was a true radial inward-flow wheel, the water turning and flowing away axially after its discharge. It had a large number of small buckets with a comparatively short distance between crown plates (or depth of runner), resulting in a wheel of rather large diameter and low speed in order to get sufficient power.

As time went on, the wheel diameter was lessened to increase the speed and the runner width increased, using a fewer number of larger buckets which extended nearer to the center of the wheel, requiring more of an axial discharge. This evolution from the Francis wheel to the American or mixed-flow wheel of McCormick and others is shown in Fig. 58,¹ and it will be noted that, as fully developed, the result was a radially inward, axial, and, to a greater or less extent (in leaving the wheel), radially outward flow.

The American type of wheel was developed particularly for use in the low-head developments (usually of 15 to 25 ft.) made invariably in this country up to about the end of the nineteenth century. Electric transmission of power about this time stimulated the use of higher heads, but the first developments of this kind were made with wheels of foreign design, as illustrated by the developments at Niagara Falls and Trenton Falls, N. Y., previously noted. The adaptation of the Francis type of runner to high-head developments by wheel manufacturers in this country proceeded rapidly, however, after about 1900, and today this type of reaction wheel is used exclusively up to the heads where an impulse wheel is needed, the present upper limit of use for reaction wheels being for a head of about 850 ft. For the low and medium heads the mixed-flow or American type of wheel now predominates, supplemented by the propeller type for low heads, as further noted.

Wheel characteristics—efficiency at both full and partial load—have also been progressively bettered, and for the last two decades American manufacturers have led the world in this respect. Efficiencies reaching 93 per cent are not uncommon, and workable values of 85 per cent and better are common where formerly only 80 per cent or less would be guaranteed. This has had a marked effect on the value of water power developments, as, when established with a market for power, any increase in efficiency of an hydroelectric plant practically means a corresponding percentage increase in income from the plant.

Propeller-type Wheels.—High speed permits the use of wheels and generators of smaller diameter for a given power output and, hence, tends to lessen cost. On the other hand, the best efficiencies of the mixed-flow reaction wheel are obtained with medium-speed types, and efficiency at part gate is also materially poorer for high-speed wheels, tending to offset the saving in cost of equipment by their use.

The propeller type of wheel is a very high-speed runner, adapted for use under low heads, up to about 50 ft., that has been developed in both the United States and Europe in several different types, all of which are essentially of the axial-flow type. In Fig. 59 are shown sectional views

¹ SAFFORD and HAMILTON, "The American Mixed-flow Turbine and Its Setting," *Trans. A.S.C.E.*, vol. 85, p. 1271, 1922.

of several of these types of wheels and also, for comparison, the modern development of the Francis runner.

The Nagler wheel, developed by the Allis-Chalmers Manufacturing Company,¹ as will be noted, is of axial-flow type. In Fig. 60 is shown a shop view of a large Nagler-type runner installed at the Green Island development of Henry Ford and Son, Inc., near Troy, N. Y. This is 156 in. in diameter, set vertically, weighs 17,000 lb., and is rated at 2000 hp. under a head of 13 ft. at 80 r.p.m. The view shows the upper or entrance side of the runner.

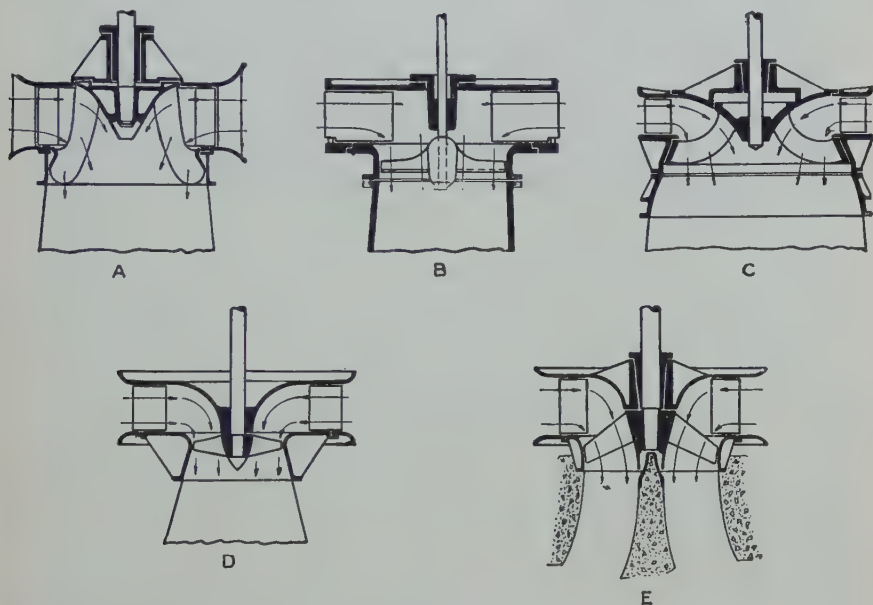


FIG. 59.—Propeller-type wheels. A, Francis. B, Kaplan. C, Dubs (*Escher-Wyss Co.*). D, Nagler. E, Moody.

The Moody diagonal type, as shown in Fig. 59, is so constructed that as the water is passing through the runner, its path is inclined to the vertical (*i.e.*, it has a radially inward component), but the water leaving is approximately axial in flow. It commonly has six overlapping blades where the Nagler type has but three or four, not overlapping. As shown, the setting is with the Moody or spreading draft tube, formed by the cone of concrete, tipped with steel.

In Fig. 61 is shown a shop view of a Moody propeller-type runner installed at the Feeder Dam development of the Moreau Manufacturing Corporation, Glens Falls, N. Y. There are five of these units, each rated at 1500 hp. under a head of 15.5 ft. at 120 r.p.m. The upper or entrance side of the runner is shown.

¹ NAGLER, FOREST, "A New Type of Hydraulic Turbine Runner," *Jour. A.S.M.E.*, December, 1919.

The Bell type or type B of the S. Morgan Smith Company was invented and developed by the Theodore Bell Company of Switzerland and the patent and manufacturing rights obtained by the S. Morgan Smith Company for the United States and Canada. It is made in two-, three-, and four-blade designs with a projected area of substantially the

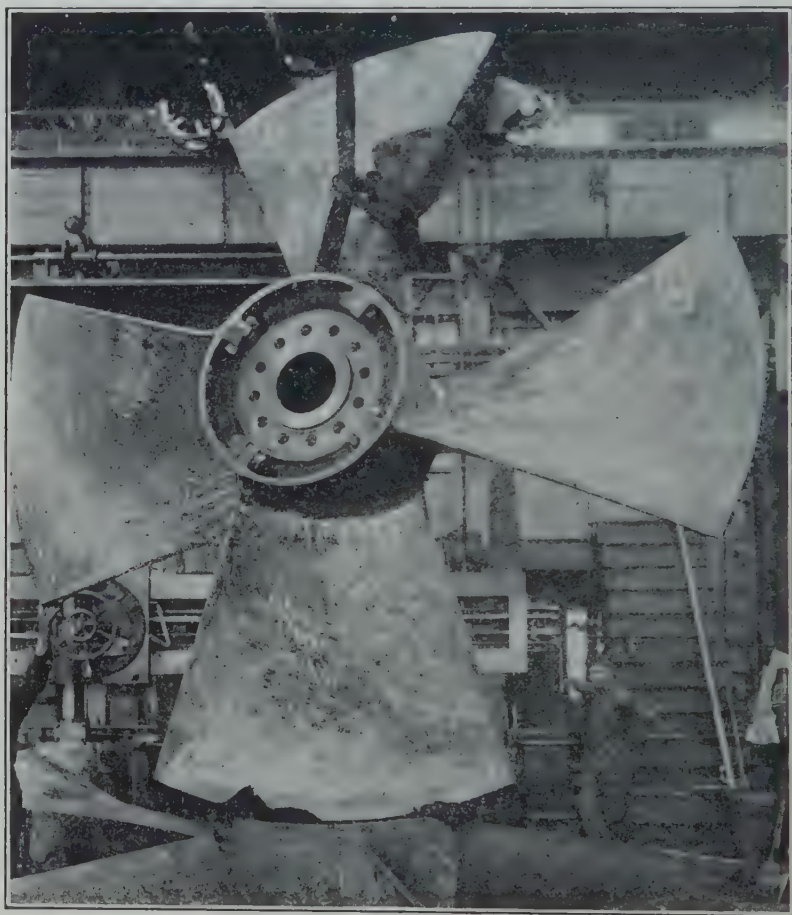


FIG. 60.—Nagler wheel runner. (Courtesy of Allis-Chalmers Mfg. Co.)

full circle. Some 24 wheels of this type have been built and are now in operation in this country. Tests at Holyoke indicate a best efficiency of 90 to 91 per cent, the four-blade runner giving the higher results.

Two European types of propeller wheels are also shown in Fig. 59, the Kaplan, an axial-flow type with few blades somewhat like the Nagler, and the Dubs (Escher-Wyss Company), more like an extreme development of the Francis type. Somewhat similar to the Dubs wheel is the type B developed in this country by James Leffel and Company,

for which a 30-in. runner, according to Holyoke test 2933, September, 1923, at full gate, gave about 253 hp. under a 15.7-ft. head at 291 r.p.m. with an efficiency of 81.4 per cent when running at the speed of best efficiency. The best efficiency, 88.8 per cent, occurred at three-fourths gate.

The new Smith-Kaplan adjustable-blade propeller-type turbine recently introduced in the country has many advantages over the older type of fixed-blade wheel. The striking feature of this wheel is its ability to adjust automatically the runner-blade angle as the wicket gates are opened or closed. Each angular position of the runner blades corresponds to a certain definite gate opening, thus maintaining best conditions of flow and eliminating shock losses at part loads. This



Fig. 61.—Moody propeller-type runner. (Courtesy of I. P. Morris Company.)

results in high efficiencies at part load, and a very flat efficiency-load curve (see Fig. 77). This wheel is of European origin but is manufactured in this country for use under heads up to about 60 ft. by the S. Morgan Smith Company.

Many installations have already been made in this country of this type of wheel. The largest powered to date are the six 42,500 hp., 220-in. diameter units at the Safe Harbor, Pa., plant completed in 1931, operating under a head of 55 ft. at 109 r.p.m. (see Fig. 62).

The Kaplan wheel has been in use abroad for some years.¹ The largest powered units are at the Ryburg, Germany, plant and are rated at 38,700 hp. under a head of about 38 ft. and are 23 ft. in diameter. These are exceeded in size, however, by the 15,000-hp., 26-ft. diameter units at the Vargon, Sweden, plant, which operate under a 14-ft. head.

¹ *Power*, July 15, 1930, p. 108 and Jan. 26, 1932, p. 127.

The S. Morgan Smith Company has also developed a wheel which occupies the range between the Francis wheel and the lower boundaries of the propeller-type wheel—their type U wheel—with a best efficiency of about 91.5 per cent.

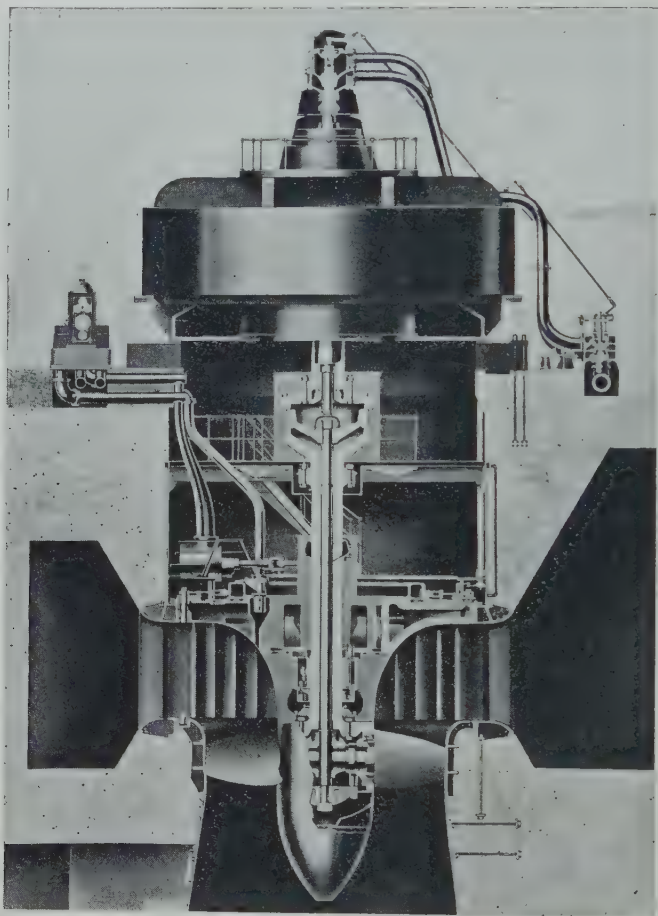


FIG. 62.—Safe Harbor Development—220-in. Kaplan adjustable blade, propeller-type turbine. (Courtesy of S. Morgan Smith Company.)

Present Wheel Manufacture.—During the last half of the nineteenth century wheel manufacturers in great number were found in this country. James Emerson,¹ who made some of the earlier tests of wheels during the period 1869–1880, gives a list of over 80 makes of wheels known by him or tested at his flumes in Lowell and Holyoke. His work was the forerunner of that of the Holyoke Water Power Company, which established its testing flume in 1882 and has, ever since, continued to do com-

¹ "Hydrodynamics," 3d ed., pp. 129–131, 1881.

mercial testing of wheels. To date, nearly 3000 tests of wheels have been made at this flume, and furnishing, as it has, the only opportunity for such tests in this country, it has played a very important part in the development of American turbines.

Comparatively few of the many wheels of Emerson's time were constructed to any considerable extent, and by 1900 there were not more than a dozen manufacturers in operation. Since that time the tendency has been more and more toward the construction of wheels adapted to fit particular conditions, rather than that of "stock wheels," and the field has been still further limited.

Following is a list of wheel manufacturers.

LIST OF MANUFACTURERS OF REACTION TURBINES IN THE UNITED STATES

Allis-Chalmers Manufacturing Company,¹ Milwaukee, Wis.

I. P. Morris Company, Philadelphia, Pa.

Holyoke Machine Company, Worcester, Mass.

Rodney Hunt Machine Company, Orange, Mass.

Hydraulic Turbine Corporation, Camden, N. Y.

James Leffel and Company, Springfield, Ohio.

Newport News Ship Building and Dry Dock Company, Newport News, Va. (formerly

Wellman-Seaver-Morgan Company, Cleveland, Ohio).

Pelton Water Wheel Company,¹ San Francisco and New York.

S. Morgan Smith Company, York, Pa.

Reaction Wheels—Essential Features.—The essential features of a modern reaction wheel (see Fig. 63, which shows a wheel setting with concrete spiral flume) are the (1) runner (or wheel, as it is sometimes called), (2) the gates and the guides, (3) the speed ring, (4) (with the vertical unit) the pit liner, (5) the casing or approach flume, and (6) the draft tube.

The runner is the portion which revolves, and consists of the vanes (or partitions separating the buckets), the crown plate, and the driving shaft. At exit from the runner passages, a shroud or band encases the wheel.

The gates control the entrance of water through the guides and usually, as here, in the form of gate wickets, actually constitute the guides.

In Fig. 64 is a shop view of a wheel runner with guides or gates in the rear. This is a high-speed runner (500 hp., 25-ft. head, 200 r.p.m.) installed at Fulton, N. Y.

The speed ring, used for concrete or steel spiral flumes and vertical units, consists of an upper and lower flange joined together by a few vanes which are shaped to conform with the flow into the guides. The speed ring is placed just outside the guides, connecting them with the casing and supporting the pitliner, which is set in the concrete walls under the generator and carries its weight. The casing or flume sur-

¹ Also make impulse wheels.

rounds the runner and guides the water to it. The draft tube connects the wheel at exit from runner with water level in the tailrace, enabling

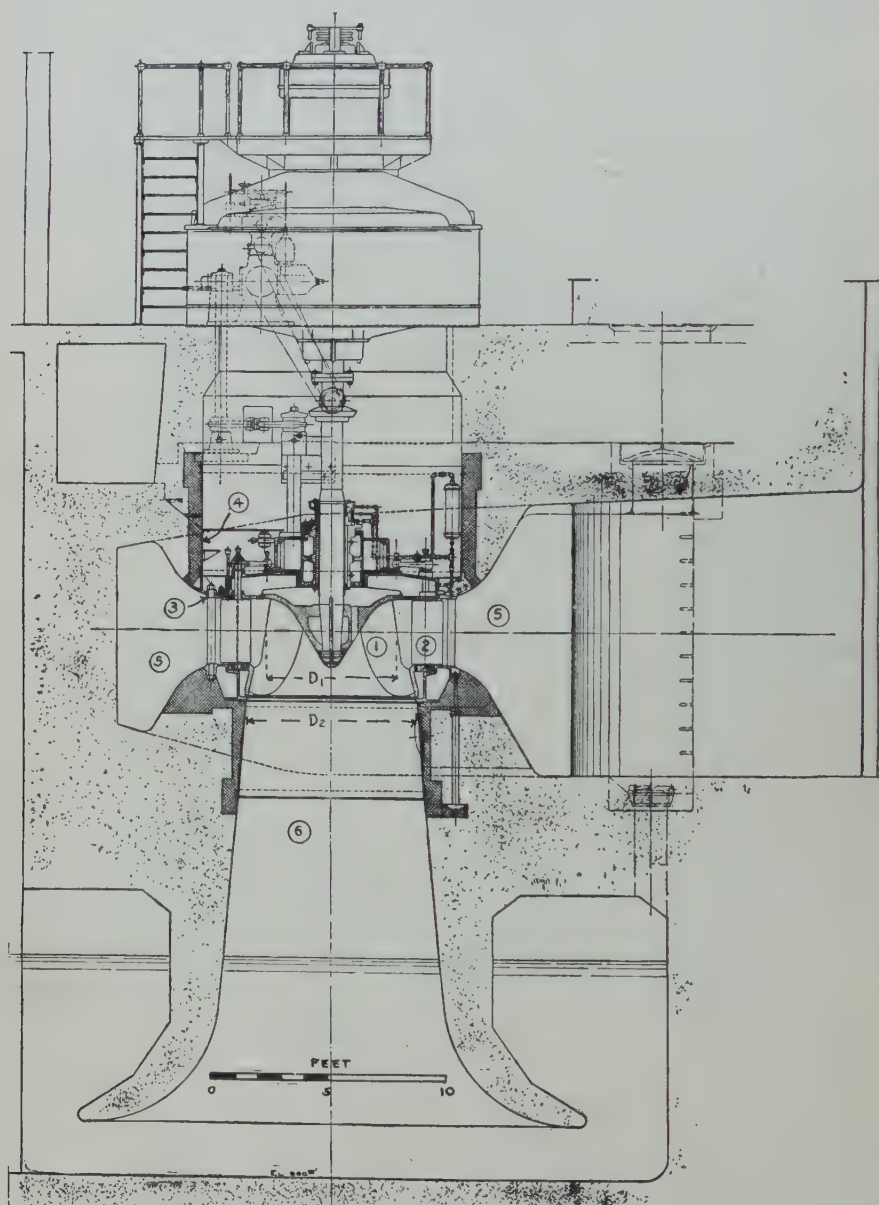


FIG. 63.—Reaction wheel—features.

the wheel to be set some distance above the latter without the loss of the head between wheel and tailrace level. The nominal or rated diameter

of the wheel, shown as D_1 in Fig. 63, is usually measured at midway of runner at entrance; D_2 is the outlet diameter or that at entrance to draft tube.

Range of Head for Reaction Wheels.—The highest head under which any reaction wheel operates, both in this country and in the world, has been, until recently, at the Oak Grove development of the Portland Railway, Light and Power Company, Oregon, where one 35,000-hp. unit utilizes a static head varying from 850 to 930 ft. The arrangement of this unit is shown in Fig. 65. A second unit of 40,000 hp. was installed at this plant during 1931. The highest head utilized by reaction wheels in this country will shortly be that at the Nantahala plant of the Nantahala

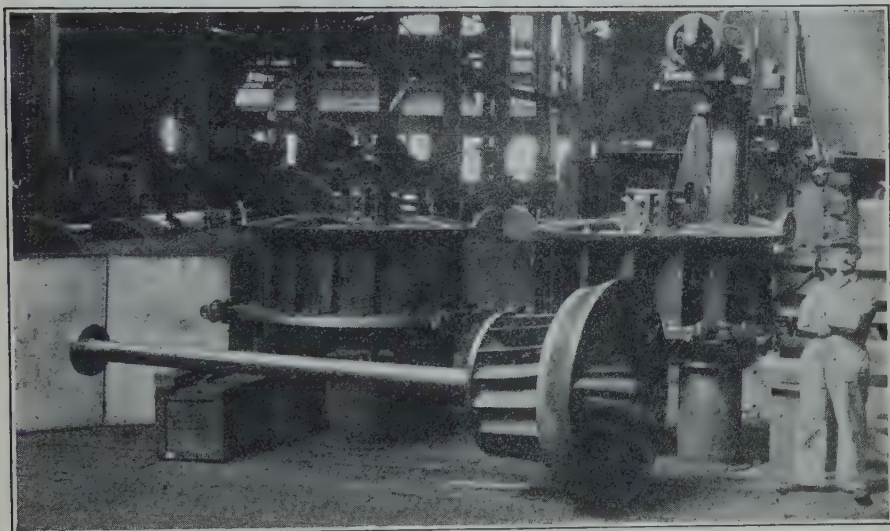


FIG. 64.—Shop view of wheel runner and guides. (Courtesy of Newport News Shipbuilding and Dry Dock Company.)

Power and Light Company in North Carolina, where one 56,000-hp., 512-r.p.m. unit is being installed under a static head of 965 ft. This head for reaction wheels is exceeded, however, by the Zappello plant in Italy, where 1180 ft. is utilized.

The lower limit of use is a few feet. A plant in Italy utilizes a head of 28 to 42 in. and one in France, of 31 in. In the latter example the turbine is placed above head-water level and requires occasional use of a steam ejector to remove air from the siphon thus formed. Usually, the limitation in lower limit of developable head is one of cost of power rather than practicability of wheel use.

Impulse Wheels.—The type of impulse wheel used exclusively in this country is the tangential or Pelton wheel, as it is commonly called. It has been developed from the crude wheels first used for water power under

fairly high heads in California as early as 1854, frequently known as "hurdy gurdy" wheels. These wheels were often of wood with flat

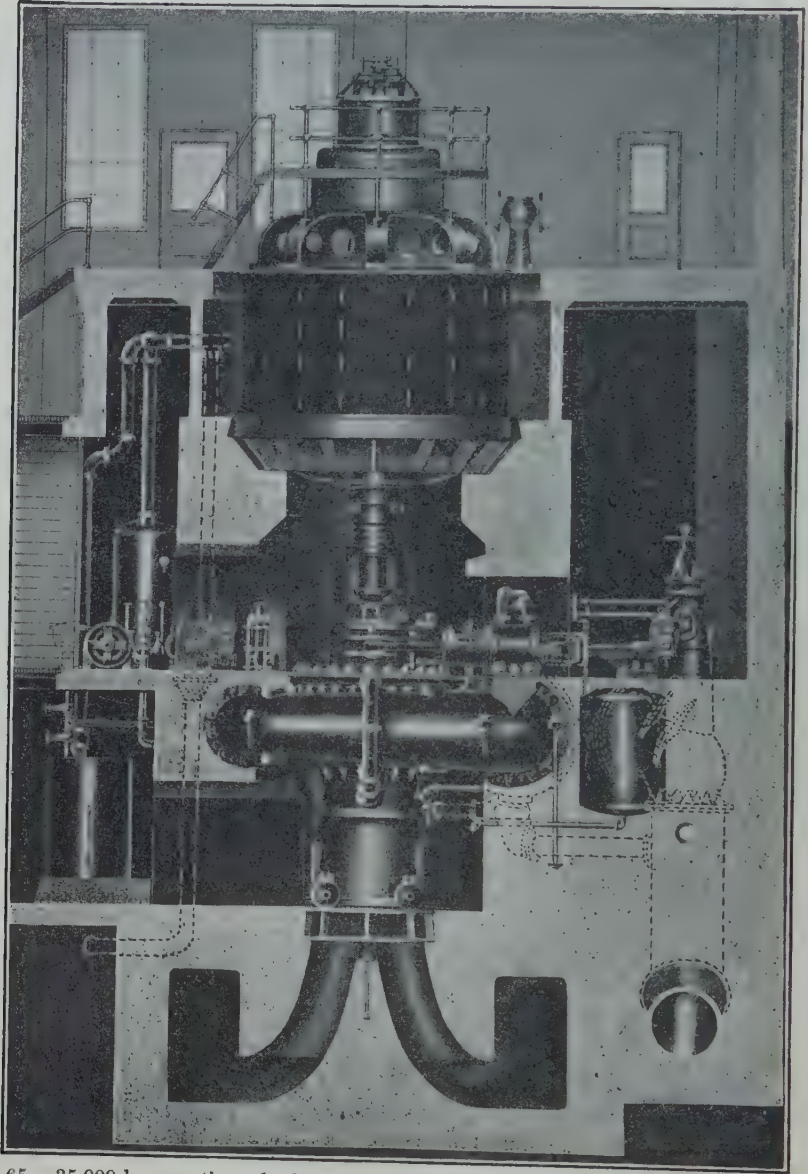


FIG. 65.—35,000-hp. reaction wheel at Oak Grove plant. (Courtesy of Pelton Water Wheel Company.)

radial plates at intervals on the circumference and gave very low efficiency. The idea of the split bucket and lateral discharge of water was

first conceived by Pelton (to whom a patent was issued in 1880); hence, the common name used for this type of wheel.

The general arrangement of the simpler form of Pelton wheel is shown in Fig. 57, page 192 and, as will be noted, has approximately rectangular buckets. The Doble bucket is ellipsoidal in shape, and this general form of bucket is now used for the larger wheels.

In Fig. 66 is a shop view of a medium-sized Pelton-wheel unit of the single-overhung type. Only two bearings are used for the entire impulse-wheel unit, whether it be of single- or double-overhung construction, the

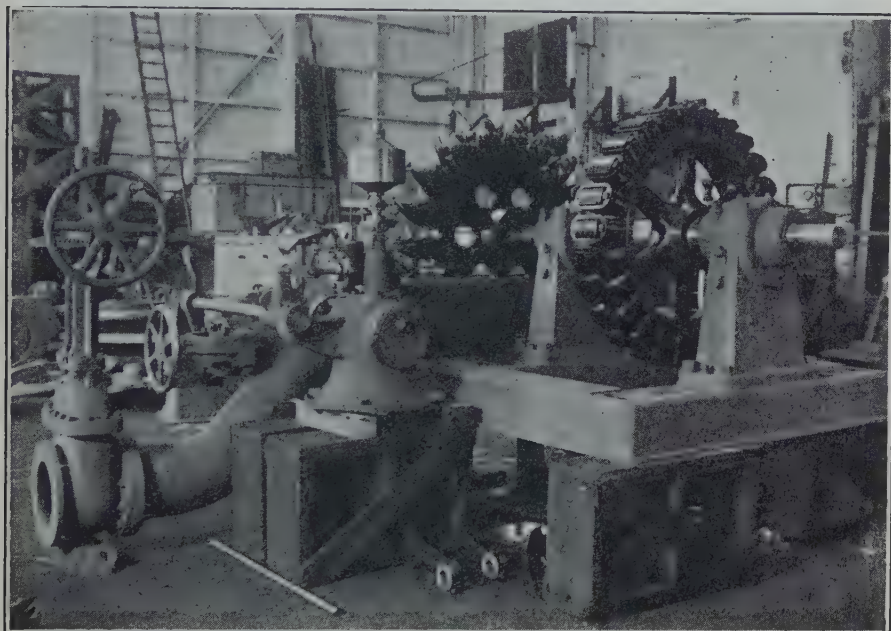


FIG. 66.—Pelton wheel shop view of single-overhung wheel. (Courtesy of Pelton Water Wheel Company.)

generator rotor being mounted between the two bearings and a wheel runner on one or both extensions of the shaft beyond the bearings.

Figure 67 shows the large impulse-wheel units installed at the Caribou plant, California, in 1921. These are of the double-overhung type, with generator between the two wheels, all on the same shaft, a common arrangement for large units. For single-wheel units, the wheel is commonly overhung in the same manner, with but two bearings.

The tangential wheel is usually set with horizontal shaft and a single jet or nozzle used. The use of two jets to increase the power of a unit is accompanied by some loss in efficiency due to their interference. For this reason, with more than two jets a vertical-shaft setting becomes necessary, an arrangement which is, however, seldom used. A notable

example of this latter type of wheel is the unit at the Salt River, Arizona, plant,¹ which has six nozzles and vertical-shaft setting. These units develop 1000 hp. under 111-ft. head at 94 r.p.m., conditions where a reaction wheel would normally have been used. The choice of the impulse wheel in this case was due to the large quantity of silt in the water, and

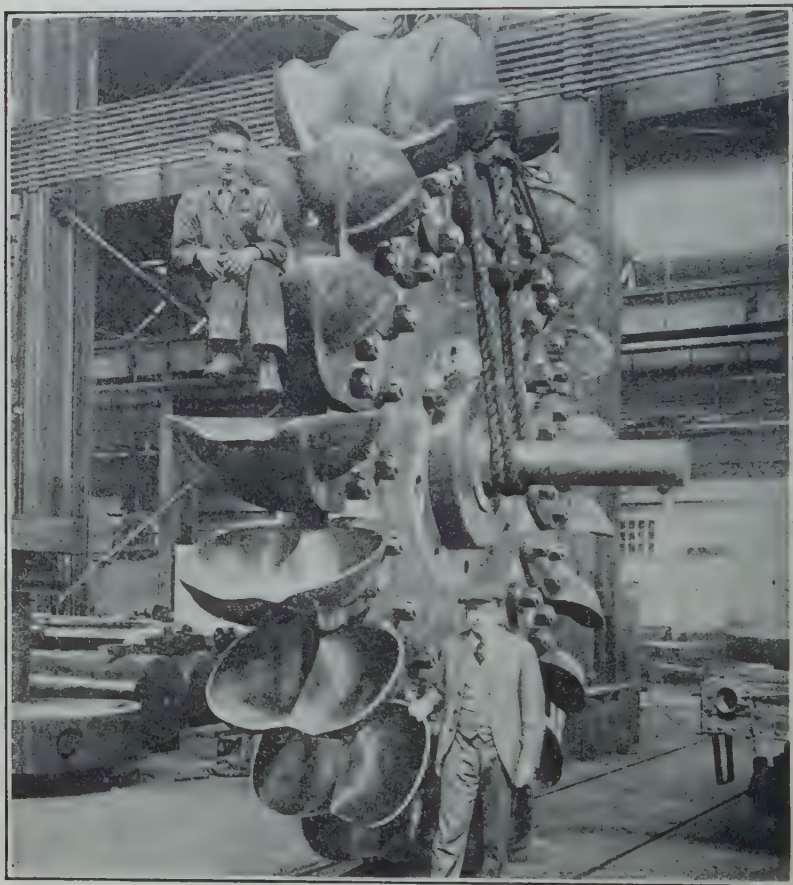


FIG. 67.—Pelton wheel—Caribou plant. Double-overhung units, 30,000 hp., 1008 ft. head, 171 r.p.m. (Courtesy of Allis-Chalmers Mfg. Co.)

consequent quick wearing out of the wheel runners, the replacement of the impulse-wheel buckets being much easier and less expensive than that of a new runner on a reaction wheel.

In Europe impulse wheels known as the Girard wheel have been used extensively. This is an outward-flow wheel on horizontal shaft with a large number of guides receiving water from a penstock under a high

¹ *Eng. News*, Apr. 20, 1916, p. 740.

pressure. Each guide acts as a nozzle, delivering a jet of water to the buckets just outside the guide. The water jets are under atmospheric pressure, however, and this is, therefore, an impulse wheel in its action, although receiving water all around its circumference. One installation of wheels of this type was made in California, but later the wheels were replaced with those of the reaction type. The Girard wheel is, therefore, not now used in this country.

Dimension drawings of a 40,000-hp. double-overhung impulse-wheel unit are shown in Fig. 68 as constructed in 1925 for the Sao Paulo Tramway Light and Power Company of Brazil, operating at 360 r.p.m. under an effective head of 2230 ft. These are similar in capacity to the wheels being installed in 1926 at the Balch plant of the San Joaquin Light and Power Corporation on Kings River, California, and were the largest impulse wheels constructed to date. The arrangement of auxiliary nozzle with curved-bucket type of baffle plate should be noted.

The efficiency of the tangential wheel is somewhat less at the full-load point than that of the best medium-speed reaction wheels. (The Caribou wheel units give about 85 per cent at the full-load point.) It holds up well, however, and usually exceeds the reaction wheel in efficiency at part gate.

The tangential wheel is essentially a low-speed wheel and becomes of use for relatively high heads, ordinarily beginning in the neighborhood of 500 ft. (unless units of very small capacity are to be used, in which case smaller heads may be utilized) and so far being limited to about 3000 ft. in this country, although a head of over 5400 ft. has been utilized by impulse wheels in Switzerland.

Size of Wheel Units.—Progress in increasing the maximum size of reaction-wheel units has been very marked during the past few years. In 1916 the Yadkin River units of the Tallassee Power Company, North Carolina, developing 31,000 hp. under 180-ft. head with 108-in. runners, were the highest in power to date. In 1920 at station 3 of the Niagara Falls Power Company, units of 37,500 hp. under 214-ft. head were installed, followed in 1922 by the 55,000-hp., 305-ft.-head wheels at the Chippewa (Niagara) development of the Hydroelectric Power Commission of Ontario, of which five are now in use. In 1924 three 70,000-hp. units with a diameter of 14 ft. 11 in. under 214-ft. head were put in operation for the Niagara Falls Power Company at their new station 3-C.

At present the highest powered wheel units in use in this country are those built by S. Morgan Smith Company and installed at the Diabolo plant of the city of Seattle, completed in 1932. These are rated at 83,000 hp. under a head of 330 ft., ranging from 247 to 347 ft. at 171.5 r.p.m. with a maximum rating of about 90,000 hp.

The wheels built for Soviet Russia for the Dnieprostroy plant by the Newport News Company, recently installed and rated at 84,000 hp.

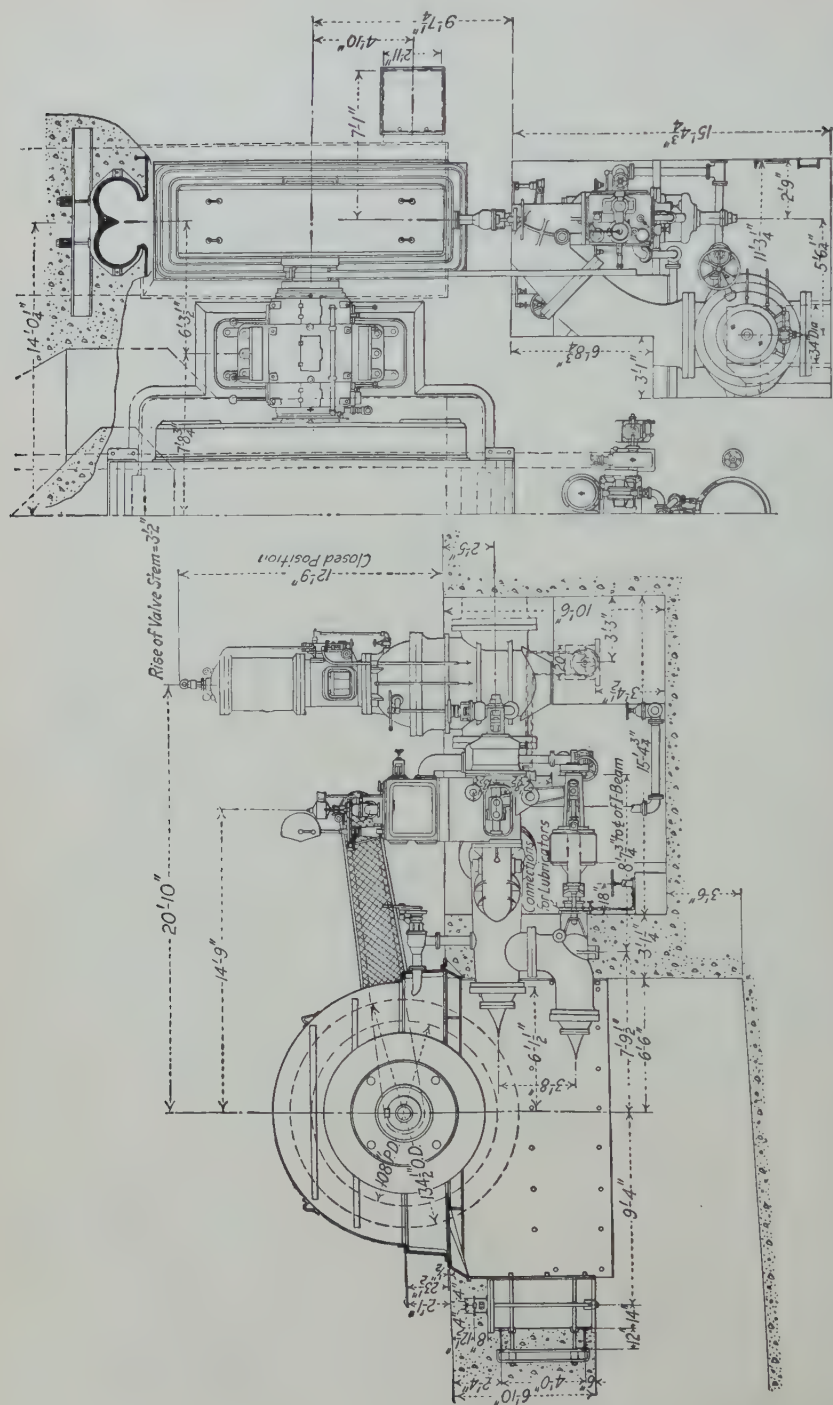


Fig. 68.—40,000 hp. Pelton wheel units for Sao Paulo Tramway and Power Company, Brazil.

(Courtesy of Pelton Water Wheel Company.)

under a head of 117 ft. at 88.2 r.p.m., are nearly as high powered as those at Seattle.

At the Boulder Dam development on Colorado River there are being installed four 115,000-hp. wheel units to be used under a maximum head of 485 ft. at 150 r.p.m.

The largest sized wheel units (other than Kaplan type)—in dimensions rather than power capacity—are the two 57,000-hp. wheels under 81-ft. head and 81.8 r.p.m. at the Spier Falls plant on Hudson River, New York, installed during 1930, with a throat diameter of 19 ft. 2 in.¹

The largest dimensioned Kaplan wheels are 15,000-hp. units under a head of 14 ft. at the Vargon plant, Sweden. They are 26 ft. in diameter.

Until recently the highest powered impulse wheels in use were those at the Caribou plant of the Great Western Power Company on the North Fork of Feather River, California. Two units began operation in 1921 and a third unit in 1924 (see Fig. 67). These units, of the double-overhung type, develop 30,000 hp. at 171 r.p.m. under a head of 1008 ft., with wheel and jet diameters of 12 ft. 4 in. and 11 in., respectively. These units were exceeded in capacity by one installed in 1925 at Big Creek plant 1 of the Southern California Edison Company, a double-overhung unit to develop 35,000 hp. at 300 r.p.m. under a head of 1900 ft.

A still larger double-overhung impulse-wheel unit, 40,000 hp., 360 r.p.m., under a static head of 2381 ft., was installed in 1926 at the Balch plant of the San Joaquin Light and Power Company on Kings River, California.

In 1929 a 56,000-hp. double-overhung impulse-wheel unit was installed, under a static head of 2419 ft. and an effective head of 2200 ft., at the Big Creek 2A plant of the Southern California Edison Company. These are the highest head units in this country.

As to the possibility of still larger units, the Pelton Water Wheel Company stated:²

The size of impulse-wheel units keeps increasing and with the improvements in the designing and control, Pelton wheels of 25,000 to 35,000 hp. are no longer considered unusual. In fact there is in contemplation at the present time a Pelton-wheel installation on the Pacific coast in which the individual capacity of the units will be 70,000 hp. The design of units of this size does not present any great difficulties with the use of present-day materials.

Practical Theory—Hydraulic Turbines. *Potential Water Power.* Opportunity for water power development exists where there is available an approximately steady river flow which may be utilized through a fall, or head H as it is commonly called. The potential available power of the water privilege in terms of flow and head is then

¹ "Civil Engineering," *A.S.C.E.* March, 1931, pp. 519–523.

² *Report of Hydraulic Power Committee, Nat. Electric Light Assoc.*, p. 128, 1924.

$$hp = \frac{QwH}{550} \quad (1)$$

If now a power development is assumed as shown diagrammatically in Fig. 69 for either an impulse- or reaction-wheel plant, it will be seen that part of the total available head H must be used up in flow through

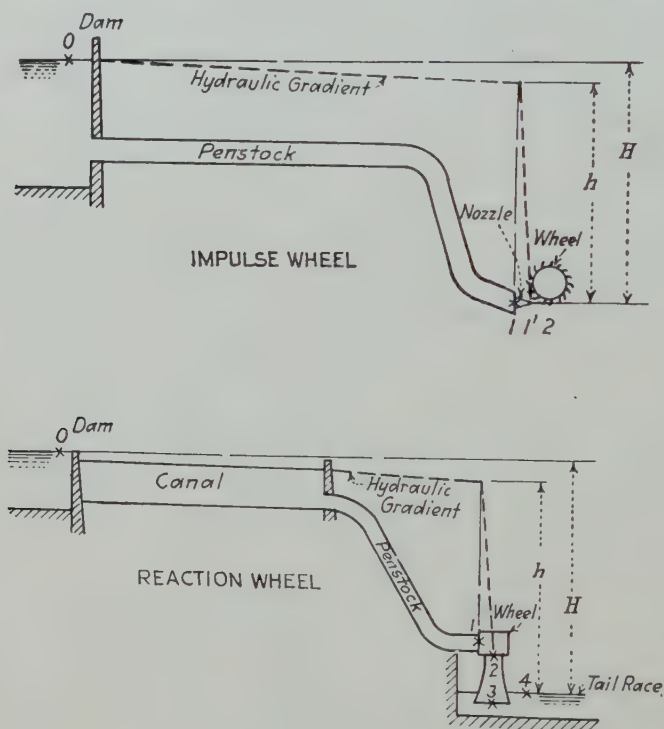


FIG. 69.—Available and effective head.

canal, penstock, etc., and the effective head h will be less than H by the amount of these losses, and the power input to the wheel will be

$$hp_i = \frac{Qwh}{550} \quad (2)$$

Power Output and Losses.—For the impulse wheel (assuming as usual the nozzle as part of the wheel) the effective head will be

$$h = \frac{V_1^2}{2g} + \frac{p_1}{w} \quad (3)$$

(assuming for simplicity nozzle and wheel shaft at a common elevation), or, in other words, the total head available at point 1 (the base of the nozzle). Between 1 and 1' occur the nozzle losses, so that at 1' the actual kinetic energy coming to the wheel corresponds to a head less

than h by the amount of the nozzle losses, often a considerable number of feet.

For the reaction wheel the effective head

$$h = \frac{V_1^2}{2g} + \frac{p_1}{w} + z_1 \quad (4)$$

considering, as is often done, that all draft-tube losses are chargeable to the wheel. Occasionally the point of view may be taken that draft-tube exit losses shall not be thus chargeable, in which case the value of h will be diminished by the term $V_3^2/2g$.

Between points 1 and 2 for the impulse wheel and 1 and 4 for the reaction wheel, in words, in going through the wheel, the effective head h is accounted for by

$$h = h_u + h_l \quad (5)$$

i.e., either as head utilized by the wheel and delivering power to the shaft or as losses. These losses consist of:

1. Hydraulic losses in the wheel, commonly expressed as a function of relative velocity at wheel exit or

$$h' = K \frac{v_2^2}{2g}$$

These are friction losses occasioned by the movement of water through the runner passages and are, therefore, for reaction wheels relatively larger for small wheels under high heads.

2. Disk-friction and Leakage Losses ($h_D + h_L$).—The first of these represents the power required to drive the runner submerged. The water above and below the runner rotates at about half the speed of the runner, and, hence, a friction loss occurs between this water and the stationary parts (the upper and lower covers) and an additional friction loss occurs between this water and the runner. To minimize these losses, the inner surface of the covers and outer surface of the wheel runner must be made smooth and free from unnecessary projections. Disk friction is relatively greater for high-head units, the power thus lost varying about according to the formula, $hp = K D^5 N^3$, and is of importance in affecting the efficiency of such units, especially at part gate.

Leakage is also of most importance with high-head units, due to escaping water between the periphery of the runner and the stationary parts. The amount of this leakage will depend upon the head and the area between the runner seals and stationary parts.

Leakage with high-head units may be minimized by the use of labyrinth seal rings for the runner, as illustrated by Fig. 70 showing such an arrangement for the 30,000-hp. units of the Big Creek plant 8 of the Southern California Edison Company, which operate under a 680-ft. head.

The use of rubber seal rings is also a recent development which permits the operation of the turbine with little or no clearance between the rotating and stationary parts and yet an elastic surface well lubricated (by water) to accommodate inaccuracies of alignment without danger of tearing or surging.

Disk-friction and leakage losses do not of course occur with impulse wheels, although the windage losses of the latter, due to air friction of the moving wheel and its parts, are somewhat analogous to disk-friction losses of the reaction wheel.

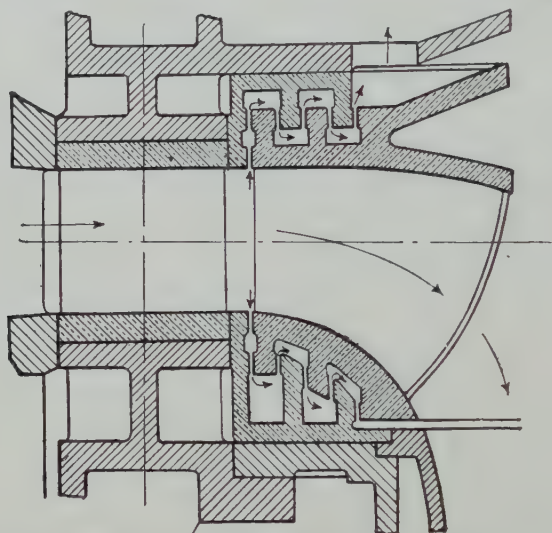


Fig. 70.—Runner labyrinth seal rings at Big Creek Plant 8, Southern California Edison Company.

3. Draft-tube and exit losses, including friction as well as the velocity-head loss at exit from the tube. These may be expressed as $h_d = m \frac{V_2^2}{2g}$. The value of m would be $(A_2/A_3)^2$ except for friction losses in the tube, which tend to make it somewhat larger and approach unity as an extreme limit.

As no draft tube is used with the impulse wheel in this case m becomes unity, or in other words, the entire velocity head at exit from the wheel is lost.

4. Shock losses, in case the wheel is running at other than full-load point or at part gate. These losses are caused by the direction of flow and its velocity at entrance to the wheel runner, being somewhat changed at part gate from the normal values for which the wheel runners are designed. These factors remain practically constant for the impulse

wheel at all wheel gates; hence, its better sustained efficiency at part gate than the reaction wheel.

Equation (5) may then be written

$$h = h_u + \left[K \frac{v_2^2}{2g} + h_D + h_L + m \frac{V_2^2}{2g} + h_s \right] \quad (6)$$

keeping in mind that $h_s = 0$ at the full-load point (or gate of best efficiency).

Not all of the head h_u utilized by the wheel effectively reaches the generator, however, as h_u is reduced by the mechanical losses, chiefly friction of the wheel on its bearings. The mechanical losses vary with the speed of the wheel N but in a somewhat irregular manner, usually between N and N^2 .

The power output of wheel to generator may be expressed in terms of input and wheel efficiency, which includes the effect of *all* the losses noted above, as follows:

$$hp = \frac{Qwhe}{550} \quad (7)$$

(Note that for 80 per cent efficiency this becomes the simple formula $Qh/11$.)

If the power is measured by friction dynamometer, as in testing the wheel, the output may also be expressed in the form

$$hp = \frac{Fu}{550} \quad \text{or} \quad \frac{T\omega}{550} \quad \text{or} \quad \frac{PD\pi N}{33,000} \quad (8)$$

where P is the effective weight on the dynamometer, taking into account its leverage ratio.

Variation in Speed of a Wheel with Head.—The linear velocity of any point on a wheel runner at some fixed gate opening and acted upon by an effective head h will vary, depending upon where this point is taken, at entrance or exit to the wheel or elsewhere, or, in other words, where the wheel diameter is measured. Practice varies somewhat in this respect but predominates in using a nominal value of D for the reaction wheel measured at about mid-depth of runner at entrance, as shown on Fig. 63, page 200, by D_1 . For the impulse wheel it would correspond closely to the center line of the buckets.

The linear velocity u of the wheel corresponding to its diameter D may be expressed in terms of the velocity corresponding to the effective head h by

$$u = \phi \sqrt{2g h} \quad (9)$$

where ϕ is a "speed ratio" or "relative-speed" coefficient having a range about as follows:

Impulse wheel — $\phi = 0.43$ to 0.47 .

Reaction wheel — $\phi = 0.55$ to 1.00 or more.

Propeller-type wheel — $\phi = 1.50$ to 3.00 or more.

The speed ratio ϕ in ordinary notation will be

$$\phi = \frac{\pi DN}{720\sqrt{2gh}} = \frac{1}{1840} \frac{DN}{\sqrt{h}} \quad (10)$$

and it will be seen that the speed N of the wheel at a given wheel gate varies as $\sqrt{h}$.

Evidently there will also be a "best value" of ϕ which may be called ϕ_e , corresponding to the speed and gate which give the highest wheel efficiency (*i.e.*, the full-load point).

Variation in Discharge and Power of a Wheel with Head.—A wheel may be looked upon as a kind of orifice and as such would follow the laws of discharge through orifices under variable head. The results of tests confirm the accuracy of this assumption and indicate that the formula

$$Q = CA\sqrt{2gh} \quad (11)$$

may be correctly used to express discharge through a given wheel under different heads.

The coefficient C is ordinarily between 0.5 and 0.6 (or not greatly different from that of the orifice) for the ordinary reaction wheel, on the basis of the area at either exit from the guides or entrance to wheel runner. C varies somewhat with the value of ϕ but usually through only a small range of variation, which can be determined from wheel tests.

The important conclusion is that the discharge of a given wheel varies as $\sqrt{h}$. Here again we may distinguish a value of C for the full-load point as C_e , corresponding to ϕ_e .

The power (output) of a wheel in terms of discharge and head will evidently be

$$hp = \frac{CA\sqrt{2gh}whe}{550} = \frac{CA\sqrt{2g}weh^{3/2}}{550} \quad (12)$$

For a given wheel, gate opening, and relative speed, all the terms except h are constant, and, hence, the power of a given wheel for a given value of ϕ varies as $h^{3/2}$.

Speed, Discharge, and Power as Affected by Diameter.—Reference to Eq. (10) indicates that for constant ϕ the wheel speed N will vary inversely as D , as the latter changes in value.

As the wheel diameter changes for a given head, variation in discharge will depend practically upon the value of A . Considering the circumferential area corresponding to D and the depth of runner B , which is

ordinarily a function of D , it will be seen that A will vary as $D \times f(D)$ or D^2 . Hence, discharge will practically vary as D^2 for a given head (and wheel gate).

Power, which varies as Qh , will also evidently vary as D^2 , for any given head and wheel gate.

Commercial Wheel Constants.—Based upon the foregoing, general formulas for speed, discharge, and power of a wheel may be written:

$$\text{Speed:} \quad N = 1840\phi \frac{\sqrt{h}}{D} = N_u \frac{\sqrt{h}}{D} \quad (13)$$

$$\text{Discharge} \quad Q = Q_u D^2 \sqrt{h} \quad (14)$$

$$\text{Power:} \quad hp = P_u D^2 h^{3/2} \quad (15)$$

An inspection of these quantities, assigning unit values to h and D , will indicate that the speed, discharge, and power coefficients, N_u , Q_u , and P_u , respectively, are the values of these corresponding factors for a 1-in. wheel acting under a 1-ft. head.

There are evidently an indefinite number of values of N_u , Q_u , and P_u for a given wheel, depending upon its gate and speed, and we may distinguish N_{ue} , Q_{ue} , and P_{ue} corresponding to the full-load point. Often, however, these constants as given for a series of wheels are for the full-gate (maximum power) rather than full-load point.

The ordinary range in value of these constants is about as follows (at full gate), based upon ordinary or nominal wheel diameters:

	Reaction wheels	Propeller types	Impulse wheels
N_u	1000-1800	2500-4000	800-1000
Q_u	0.001-0.05	0.015-0.025	0.0002-0.0005
P_u	0.0001-0.005	0.0015-0.0025	0.00002-0.00005

Values of N_u and P_u for certain selected wheels are also given in Table 56, page 215.

It will also be noted that $P_u = Q_u \times \frac{62.5}{550} \times e$, or if e is 80 per cent, $P_u = Q_u/11$, and ordinarily P_u is between $1/10$ and $1/11$ of Q_u .

Specific Speed.—Equation (13) enables the selection of the size of wheel required for a given value of N ; Eq. (15) the selection of size of wheel to give a certain horse power. These equations are independent, however, and to get suitable values of N and hp requires successive solutions of them. By combining these equations, however, a single equation may be obtained taking into account both speed and power as follows: From Eq. (13):

$$D^2 = N_u^2 \frac{h}{N^2}$$

From Eq. (15): $hp = P_u D^2 h^{3/2} = P_u \frac{N_u^2 h}{N^2} h^{3/2}$

whence
$$\frac{N \sqrt{hp}}{h^{3/4}} = \sqrt{P_u} N_u = N_s \quad (16)$$

The significance of N_s , called specific speed, may be seen by assigning unit values to hp and h , under which condition $N = N_s$ or, in other words, the specific speed is the speed of a (hypothetical) wheel of the series, when delivering 1 hp. under 1-ft. head. Note also that $N_s = \sqrt{P_u} N_u$, a useful relation for computations.

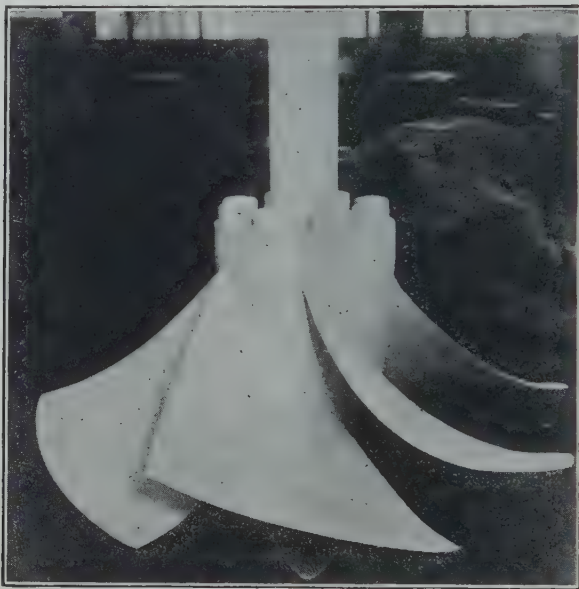


FIG. 71.—Type B wheel runner. (James Leffel & Co.)

Specific speed is a very useful wheel constant. As given, without qualification, it applies (as in the case of the other wheel constants N_u , Q_u , and P_u) usually to the full-gate point (*i.e.*, maximum power under normal speed) but occasionally is given for the full-load point or the gate for best efficiency. The ordinary range of values of N_s is as follows:

Impulse wheel.....	3 to 6
Reaction wheel.....	10 to 110
Propeller types of runners.....	110 to 225

Illustrative Values of Wheel Constants.—In Table 56 for wheels of various different manufacturers, values are given for the wheel constants N_u and N_s , arranged for each make of wheel in regard to values of N_s . Data as given are taken in part from catalogues or memoranda of the various wheel manufacturers and partly from the results of wheel tests

at Holyoke, furnished through their courtesy. As will be noted, a range of N_s from about 31 to 172 is covered for reaction (and propeller-type) wheels, with data also for impulse-wheel units, both single and double overhung.

TABLE 56.—TURBINE CHARACTERISTICS

Number	Make	Diameter, inches	Type	Holyoke test ^a number	Full-gate constants		
					N_s	P_u	N_u
1	S. Morgan Smith.....	44	E	2288	31	0.000542	1330
2	S. Morgan Smith.....	47	F-1		38	0.00080	1390
3	S. Morgan Smith.....	53	H	2369	54	0.00140	1440
4	S. Morgan Smith.....	30	K	1983	63	0.00198	1410
5	S. Morgan Smith.....	56	N		73	0.00275	1390
6	S. Morgan Smith.....	26	S		85	0.00408	1380
7	S. Morgan Smith.....	9.4	U		106	0.00540	1440
8	S. Morgan Smith.....	116	B		142	0.00165	3500
9	S. Morgan Smith.....	60	Kaplan		152	0.00237	3100
10	Newport News.....	45	W.S.M.	2674	37	0.000742	1330
11	Newport News.....	30.75	W.S.M.	2846	58	0.00163	1440
12	Newport News.....	27.5	W.S.M.	2498	69	0.00235	1420
13	Newport News.....	25	W.S.M.	2403	85	0.00338	1450
14	I. P. Morris.....	26.8	M-I.P.M.	2127	65	0.00196	1450
15	I. P. Morris.....	35	No. 81, Exp.	2862	111	0.00156	2790
16	I. P. Morris.....	16	No. 68-D-6		149	0.00198	3380
17	Allis-Chalmers.....	41.5	C-D	2902	36	0.00078	1280
18	Allis-Chalmers.....	30.2		2959	70	0.00266	1350
19	Allis-Chalmers.....	24		2592	100	0.00428	1520
20	Allis-Chalmers.....	156	N.X.		144	0.00176	3430
21	Allis-Chalmers.....	34	N.X.	2953	146	0.00218	3140
22	Allis-Chalmers.....	98	N.X.		162	0.00209	3600
23	Allis-Chalmers.....	34	N.X.	2905	172	0.00228	3600
24	Leffel.....	35.5	F	2359	83	0.00233	1710
25	Leffel.....	30	Z	2683	101	0.00481	1450
26	Leffel.....	30	B	2933	148	0.00444	2210
27	Rodney Hunt.....	48	A		46	0.00119	1330
28	Rodney Hunt.....	33	B		50	0.00165	1230
29	Rodney Hunt.....	33	C		61	0.00200	1360
30	Rodney Hunt.....	33	E		71	0.00224	1490
31	Rodney Hunt.....	24	HS	2973	101	0.00575	1330
32	Pelton.....	140	Impulse		3.42 (a)	0.000011	1030
33	Pelton.....	90	Impulse		4.32 (a)	0.000027	834
34	Pelton.....	108	Impulse		4.67 (b)	0.000033	765
35	Allis-Chalmers.....	156	Impulse		5.30 (b)	0.000038	855

(a) Single-overhung unit. (b) Double-overhung unit.

To compare these wheels better and show the range of available power and speed in Fig. 72 a plot is shown of P_u against N_s , for data as given in Table 56, as well as data for many additional wheels. While there are

some individual variations, it will be noted that a good average or ordinary curve for this relation (curve A) may be drawn as shown for American-type wheels and another approximate curve B for propeller types.

It will be noticed that in Fig. 72 the Leffel wheel, $N_s = 148$ (type B) appears to have a much higher value of P_u than the propeller-type wheels

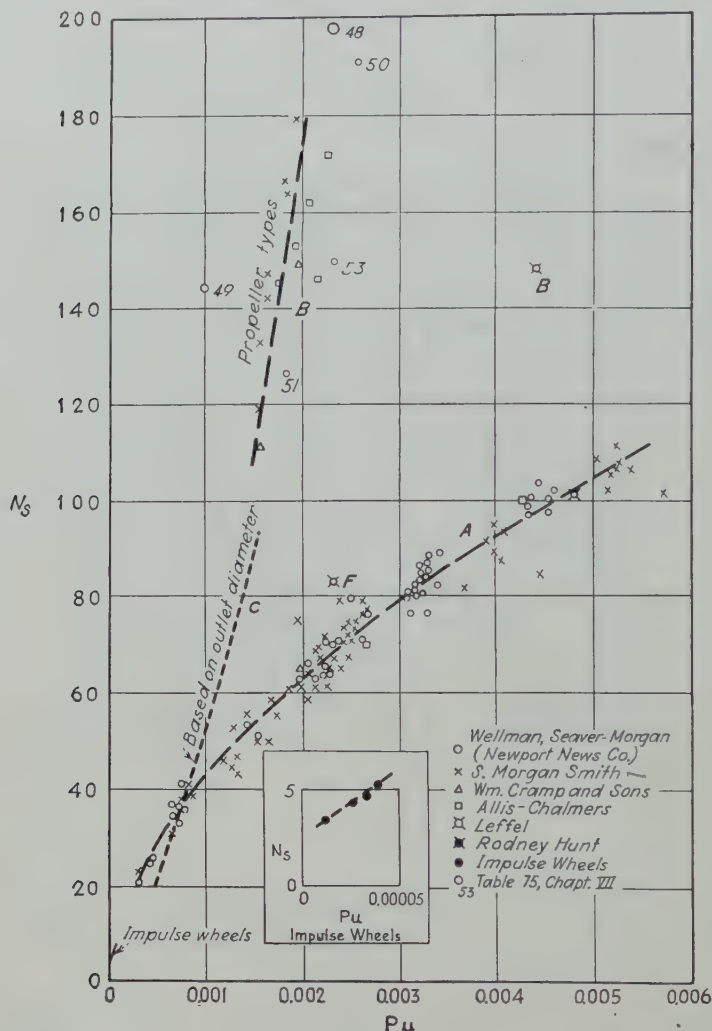


FIG. 72.—Relation $N_s - P_u$.

of this specific speed. This is due to the shape of the Leffel type B wheel, (see Fig. 71) which is like a cutaway Francis or mixed-flow wheel such that the rated diameter of the wheel is not comparable with the other wheels on this diagram. A more consistent curve of the $N_s - P_u$ relation for the whole range of N_s from 20 to 175 can in fact be obtained by com-

puting P_u , based upon outlet diameter, instead of rated diameter. Such a curve is shown by the dotted line curve C at the left in Fig. 72, where are plotted values of P_u based upon outlet diameter of wheel for the wheel units listed in Table 75, Chap. VII, including five propeller-type wheels. Curves B and C may with slight change be combined in one average curve which includes all types of reaction wheels up to $N_s = 180$ approximately.

For specific speeds in the vicinity of 100 to 110, the propeller type shows a slightly less power coefficient than the ordinary reaction wheel. Considering the turbine as a form of orifice, it will be seen that for a given area at outlet there should be a similar discharge and power for any given head (at say full gate) provided efficiencies and discharge coefficients are the same. As the propeller type is usually somewhat less in efficiency than the ordinary reaction wheel, the power coefficient of the former is likely to be somewhat less, although its coefficient of discharge is perhaps a little greater, somewhat offsetting the lesser efficiency.

It is also to be noted in Fig. 72 that for low specific speeds, about 20 to 30, the power coefficients based upon outlet diameter, curve C , are somewhat greater than for nominal or entrance diameter, curve A , as wheels of this specific speed are of the strictly Francis type, with the entrance diameter a little larger than outlet diameter.

A curve of the $N_s - P_u$ relation for impulse wheels is shown in Fig. 72 to a larger scale, and it will be seen that with this type of wheel a fairly definite relation is also shown. Furthermore, this curve when placed on the main diagram is consistent with curve A in its direction. The gap in curve A between about $N_s = 6$ and $N_s = 20$ represents a field not covered by American wheels.

Factors Affecting Specific Speed.—Inspection of Eq. (16) will indicate that N_s varies directly as N and $\sqrt{hp}$, and inversely as $h^{5/4}$, which latter value may be conveniently computed as $h\sqrt{\sqrt{h}}$. To ascertain the effect of this power of h note the following:

Item	$h = 10$	$h = 100$	$h = 1000$
$h^{5/4}$	17.7	314	5600
Ratio $\frac{h^{5/4}}{h}$	1.77	3.14	5.60

For ordinary values of h this term is, therefore, usually from about 2 to $4h$.

In Table 57 the effect of varying the individual factors in the equation for N_s is shown, from which the following is evident:

1. The dominant factor is the very high head, 1600 ft., resulting in a low value of N_s furnished only by a wheel of the impulse type. A smaller r.p.m. could be used with larger units. Thus, doubling the capacity to 4000 hp. would require $N = 1000/\sqrt{2} = 710$ to keep the same value of N_s .

2. Head is rather low and capacity fairly high, resulting in a high value of N_s , a typical situation for a low-head plant.

TABLE 57.—VARIATION IN SPECIFIC SPEED—(N_s)
With hp , h , and N

Number or plant	hp	h	R.p.m.	N_s
Assumed installations				
1	2,000	1,600	1,000	4.42
2	5,000	36	100	80.3
3	12	36	100	3.95
4	12	36	600	23.6
5	10,000	60	300	179.5
(4 units)	10,000	60	300	90
	4			
Actual installations				
Caribou, California.....	30,000	1008	171	5.2 ^(a)
Salmon River, New York.....	10,000	275	375	33.8
Cedars.....	10,800	30	54.3	81
Yadkin River, North Carolina.....	27,000	165	154	43
Chippewa, Niagara.....	55,000	305	187.5	34.7
Niagara Falls Power Company, 1923.....	70,000	213.5	107	34.7
Vernon, New England Power Company old units, three runners, 1909.....	3,200	34	133	91
Vernon, new single-runner units, 1923.....	4,200	34	133	105
Green Island plant, Nagler type.....	2,000	13	80	144
Great Falls, Winnipeg River, Moody-propeller type.....	28,000	56	138.5	153
Dnieprostroy, Dnieper River, Russia, 1929...	84,000	116	88.2	67
Safe Harbor, Pa. (Smith-Kaplan).....	42,500	55	109.1	151

(a) Double-overhung impulse wheels, N_s for one wheel = 3.7.

3. The head is still low, the speed the same as before, but the capacity so small that again a wheel of the impulse type is required. This would of course be a very unusual installation, a very small wheel.

4. By increasing the speed sufficiently, even with this small capacity a low-speed reaction wheel is indicated.

5. The dominant factor is the high capacity, 10,000 hp., resulting in a specific speed beyond the range of the ordinary mixed-flow wheel. To keep this capacity, N might be made 150 and $N_s = 90$, or, with four units as noted, the specific speed would fall to 90, which is possible for this type of wheel. The conditions as first assumed might perhaps be met by a wheel of the propeller type.

Data from certain actual installations are also given in the table. Of particular interest are the very large units at Niagara and the propeller-

type runners. Also note that the old three-runner units at Vernon ($N_s = 91$) are being replaced with single-runner units ($N_s = 105$) of materially greater specific speed, illustrating progress in this direction since 1909.

Study of Turbine Characteristics.—As a basis for the study of turbine characteristics it is necessary to have what is called a complete wheel test,

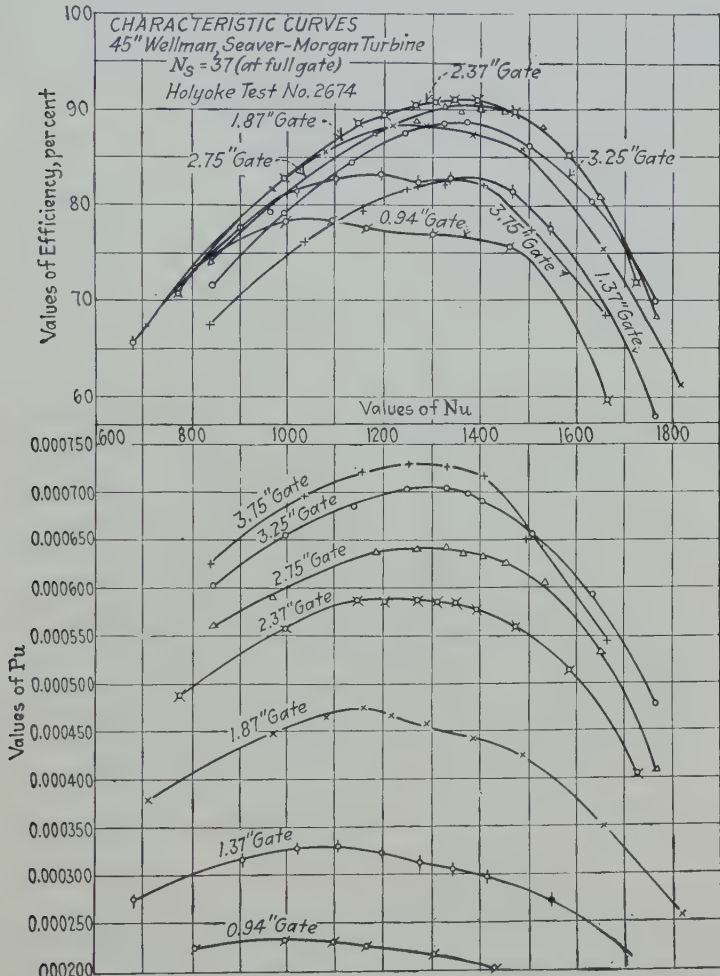


FIG. 73.— $N_u - P_u$ and efficiency. Low-speed wheel.

viz., determinations of power and efficiency at various speeds and for different gate openings, or, in other words, such information as is obtained and furnished by the Holyoke test of a wheel. A water turbine must run at practically constant r.p.m. to accord with generator speed, which is fixed within close limits. It is often desirable, however, to study the characteristics of a wheel in order to select the best r.p.m. This would

commonly be the r.p.m. for the point of highest efficiency given by the wheel, which would ordinarily mean at a setting in the vicinity of three-fourths gate. As previously stated, this is often called the "full-load" point of the wheel, which may be thus defined as the point of maximum efficiency under normal or best speed.

By opening up the wheel gate beyond the full-load point, more power will be developed up to full gate, which may be defined as the gate opening for maximum power, under normal speed.

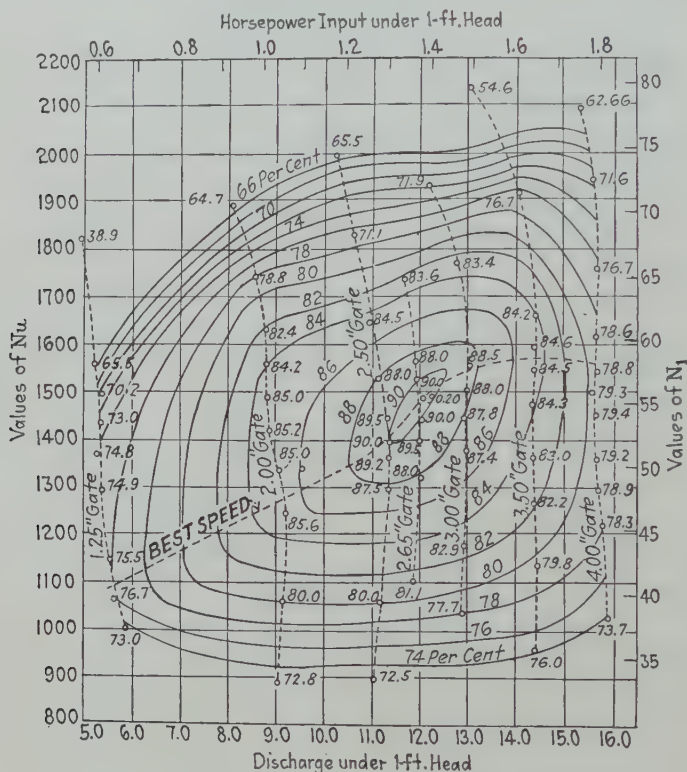


FIG. 74.—Characteristic curve—medium-speed wheel.

To determine the power and efficiency at other than the full-gate point, a plot of two sets of curves showing N_u as abscissas and (1) b.h.p. and (2) efficiency as ordinates is useful. Such a set of curves is shown on Fig. 73 for a 45-in. Wellman-Seaver-Morgan wheel, based upon Holyoke test 2674. The full-load point is at about 2.37-in. (or 0.63) gate, with an efficiency of 91.0 per cent at $N_u = 1360$. At this (normal) speed full-gate (3.75 in.) power corresponds to $P_u = 0.000723$. If desired, instead of N_u , values of N_1 (the r.p.m. of the 45-in. wheel under 1-ft. head) may be plotted as abscissas, or again values of ϕ (which is $N_u/1840$). Note the variation in speed for best efficiency as gate opening changes; also

that with this type of wheel (medium-low N_u) the power output at full gate decreases as N_u increases in value above the normal of 1360. Such a diagram furnishes in convenient form all the necessary information for studying the characteristics of the wheel.

Another useful form of diagram is what is known as the "characteristic curve" as shown in Fig. 74, which gives on one diagram or set of curves complete information with regard to wheel characteristics. As will be noted, this is a plot of discharge and horse-power input (or any function

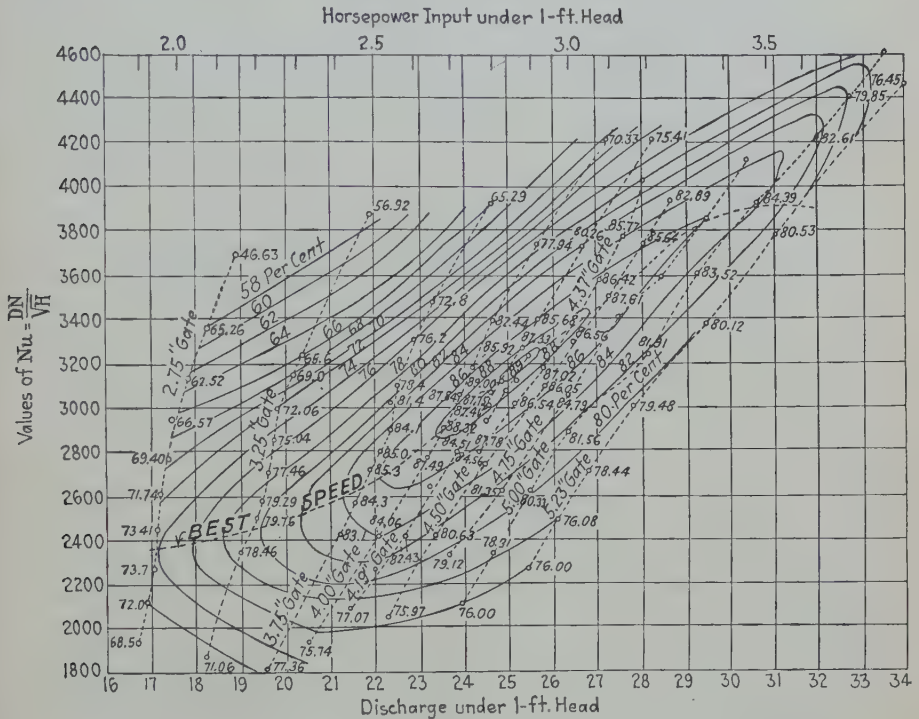


FIG. 75.—Characteristic curves for 34-in. Allis-Chalmers Type NX wheel.

of discharge) under 1-ft. head as abscissas against any function of speed as ordinates, such as ϕ , N_u , r.p.m. under 1-ft. head or r.p.m. under the actual head of installation, etc. These curves are made up from the individual test points of the Holyoke test by plotting them in their position with reference to ordinates and abscissas, and marking each one with its efficiency. The wheel-gates lines are shown as dotted lines connecting these plotted points, and lines of equal efficiency are also interpolated between them. The resulting plot is analogous to the contour map of a hill, the point of highest efficiency (full-load point) being the summit of the hill. Furthermore, the dotted line marked "best speed," connecting the points of best efficiency at each gate, clearly shows the

effect of variation in speed from the normal for different wheel gates. The characteristic curve may be still further developed for convenience in use by plotting lines of horse-power output in color.

The characteristic curves in Fig. 74 are for a medium speed ($N_s = 65$) I. P. Morris wheel, based on Holyoke test 2127. Full-load point is at 2.65-in. (0.66) gate with a best efficiency of 90+ per cent and $N_u = 1475$. Lines of horse-power output are not shown.

In Fig. 75 is shown a characteristic curve of a propeller-type wheel ($N_s = 146$), which is markedly different from the medium-speed wheel ($N_s = 65$) in its features.

While the "hill" of efficiency for the latter wheel is approximately rounded in all directions, that for the propeller type is more like a long ridge and requires more change in speed to keep the best efficiency for the

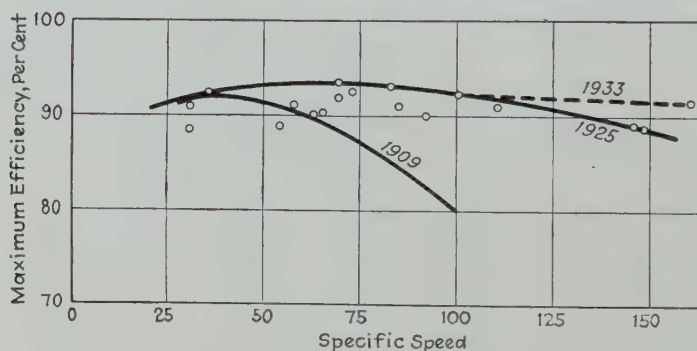


FIG. 76.—Specific speed-efficiency.

propeller wheel as gate changes. Moreover, for the latter the summit of the "hill" is lower and its slopes steeper than for the medium-speed wheel (Fig. 74).

At the end of the chapter are given Holyoke test data of two Allis-Chalmers wheels 30.2 and 34 in. in diameter, with values of N_s of 70 and 172, respectively, for further study by the methods outlined previously.

Specific Speed and Efficiency.—To show the relation between specific speed and best wheel efficiency a plot may be made as in Fig. 76, with specific speed as abscissas and efficiency at the full-load point as ordinates. The upper curve marked 1925 is a limiting curve for the various plotted points, representing several different makes of wheels, based upon Holyoke test data. The curve marked 1909 is as given by Prof. L. F. Moody in his discussion of Larner's paper on "Characteristics of Modern Hydraulic Turbines"¹ and represents the best practice of that date.

As will be noted, since 1909 there has been some gain in the peak of efficiency, now about 93 per cent, as compared to 92 per cent at the former time. The noteworthy difference is, however, the sustaining of relatively

¹ *Trans. A.S.C.E.*, vol. 66, p. 348, Plate XXXII, 1910.

high efficiency for the higher specific speeds and the extension of this field, particularly by the Smith-Kaplan wheel, to $N_s = 150$ and beyond. Whereas the best efficiency was formerly at about $N_s = 30$ to 40, it is now from about $N_s = 60$ to 75.

At low specific speeds (high-head wheels), the bucket passages are relatively smaller, and hydraulic friction losses thereby become greater. The effect of leakage through the clearance spaces between runner and guides is also relatively larger with low-speed wheels, due to the relatively small wheel passages. On the other hand, for high specific speeds—above 80—greater discharge losses at runner exit tend to lower the efficiency. This last element of loss has been minimized somewhat by greater care in the design of wheel settings and draft tubes, but, all things

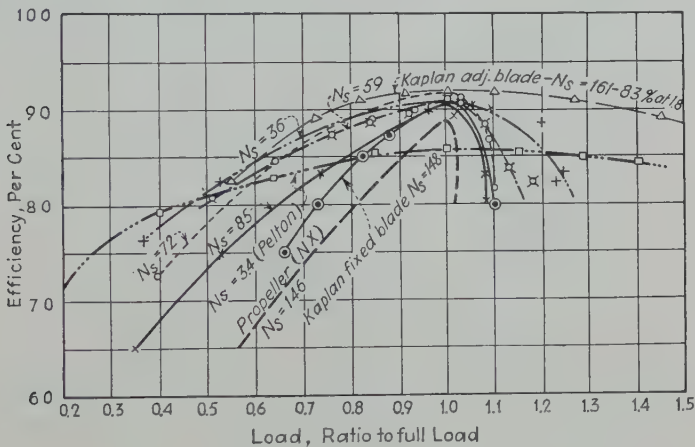


FIG. 77.—Load-efficiency.

considered, the highest efficiencies will be obtained with medium-speed wheels.

Efficiency at Part Load.—The foregoing statements were for efficiency at the full-load point or the peak of efficiency. At part load, as well as for loads greater than that at the most efficient gate, the efficiency will be less than the maximum, and it will fall off at different rates, depending upon the type or specific speed of the wheel.

As previously noted, due to the absence of shock losses, the impulse wheel has a high relative efficiency at part gate. For reaction wheels, part-gate efficiency is relatively lower as specific speed increases. Thus the low-speed wheel would not differ greatly from the impulse wheel as to relative efficiency at part gate; the efficiency of the high-speed wheel ($N_s = 90$ to 110) falls off fairly rapidly for loads other than at the best gate, and the fixed-blade propeller type of wheel ($N_s = 120$ to 175 or more) has a very sharp peak to its load-efficiency curve while the Kaplan adjustable-blade propeller-type wheel shows the highest efficiency of any

type of wheel at part gate. In Fig. 77 are shown such curves for a number of reaction wheels with different values of N_s , including a Kaplan fixed-blade propeller-type wheel ($N_s = 148$), an adjustable-blade Kaplan ($N_s = 161$), and also a Pelton (impulse) wheel ($N_s = 3.4$). In each case the speed is assumed to be maintained at the r.p.m. which gives the best efficiency at the full-load point. The relatively flat curve for the Pelton wheel, or well-sustained efficiency, is evident, although the maximum efficiency—about 85.5 per cent—is considerably less than that for the reaction wheels, which all exceed 90 per cent. The Kaplan adjustable-blade propeller-type wheel shows an even flatter load-efficiency curve than the Pelton wheel—with a remarkably well-sustained efficiency at part load.

In Figs. 78 and 79 are further curves based upon those in Fig. 77, as well as some not there shown. In Fig. 78 are plotted peak efficiencies

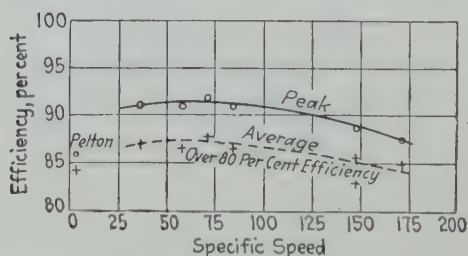


FIG. 78.—Peak and average efficiencies.

and average efficiency above the 80 per cent line. The latter, as will be noted, is approximately 4 per cent less than the peak efficiency through the entire range of N_s . This average efficiency is more truly representative of the working characteristics of the wheel than the peak efficiency, as it is seldom possible to hold the wheel gate steadily at the best point.

In Fig. 79 are curves showing the range of load covered by the 80 per cent line of efficiency in Fig. 77, this load range being defined by a curve of "upper load limit" varying from about 1.3 to 1.05 load and a "lower load limit" varying from about 0.4 to 0.8 load—all with reference to the full-load point as 1.0. As specific speed increases, this load range decreases in amount to the small range typical of the fixed-blade propeller type of wheel.

The area between the load-efficiency curves in Fig. 77 and the horizontal line at 80 per cent efficiency is an index of the manner in which efficiency is sustained, and is shown in the lower curve in Fig. 79, called "efficiency-load range factor." The values for this curve were obtained from the product of the average efficiency above the 80 per cent line and the range of load. While the lowering in peak of efficiency for the higher specific speeds is at the most only 3 or 4 per cent, as compared to the best values at $N_s = 70$, the efficiency-load range factor decreases rapidly as N_s

increases, reaching about 0.15 for fixed-blade propeller-type wheels ($N_s = 175$), as compared to 0.7 for low-specific-speed reaction wheels and 0.8 for Pelton wheels. The Kaplan adjustable-blade wheel with N_s about 150 and an efficiency-load range factor of about 1.5 (or above the limits of the plot in Fig. 79 greatly exceeds any other type of wheel in this respect.

The efficiency-load range curve shows clearly the comparative working characteristics of wheels of different specific speeds, as it not only includes the effect of their efficiency peak but also the extent to which efficiency is maintained over the common range of load. Some base line other than 80 per cent—such as 75 or 70 per cent—might have been used, but the relative results would have been about the same.

Overgate with High-speed Wheel.—Full gate has been defined as the point of maximum power, under normal speed. In the case of a low-head

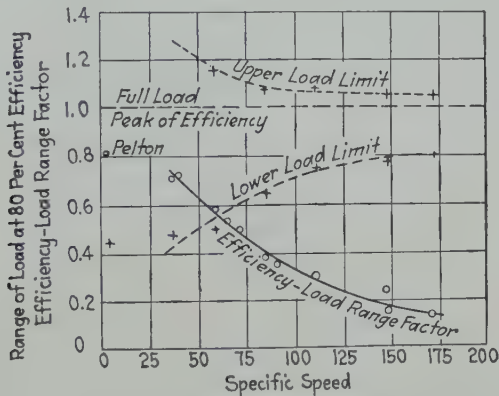


FIG. 79.—Load limits and efficiency-load range factor.

development, as on a large river, as previously noted (see page 165), during the high-water season the head may be reduced below the normal due to backwater in the tailrace. An inspection of Eq. (13) will show that under these conditions the value of ϕ or N_u upon which the wheel is running will be greater than the normal, N necessarily being kept constant. By reference to Fig. 80, on which are shown the characteristics for a high-speed runner, it will be noted that as N_u increases above the normal, the power of the wheel at full gate increases somewhat. Furthermore, by using an overgate, that is, opening the wheel somewhat farther, the power is still further increased. (Note that at normal speed the use of overgate does not increase the power.) It will be seen, therefore, that a wheel of this type is of value for a low-head development, subject to backwater, due to this overgate feature, which helps to make up some of the deficiency in power due to the lessened head under such conditions.

An inspection of the characteristics of a medium- or low-speed runner (see Fig. 73) shows no such increase in power (but often a decrease) with

overgate under increased N_u or ϕ , and this feature of increased power at higher relative speeds is essentially one pertaining to high-speed wheels. It is due to the fact that the high-speed wheel discharges water more or less outward, which discharge is aided by the speed of rotation of the runner, and increases as this speed increases. The low-speed runner is

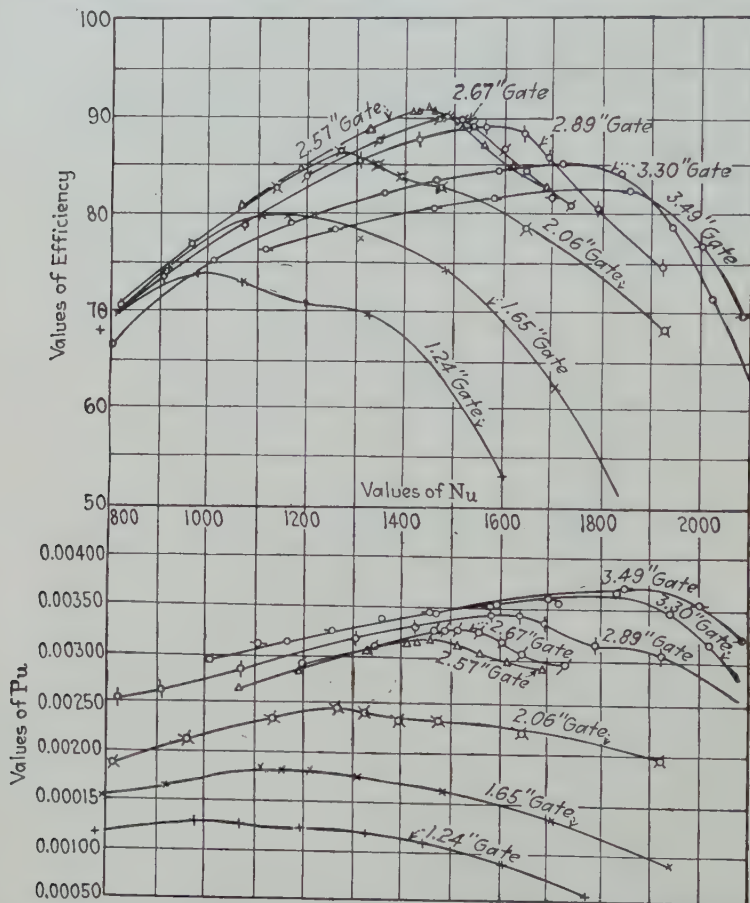


FIG. 80.— $N_u - P_u$ and efficiency. High-speed wheel (25-in. Wellman-Seaver Morgan).

substantially an inward-flow wheel, and its discharge is opposed by the rotation of the runner and, hence, tends to decrease as a rule as speed increases. The propeller-type runners, discharging axially, do not have this overgate feature.

*Type of Wheel as Affected by Head.*¹—Considering first low-head plants, it will be seen that the high-speed runner is best adapted for such installa-

¹ See LARNER, "Characteristics of Modern Hydraulic Turbines," *Trans. A.S.C.E.*, vol. 66, pp. 343-346, 1910; also "Types of Runners for Varying Heads," *Report of Hydraulic Power Committee, Nat. Electric Light Assoc.*, pp. 120-121, May, 1924.

tions. As the speed of a wheel varies as $\sqrt{h}$, the inherent tendency under a low head is toward a low actual r.p.m. Hence a type of wheel which will counteract this effect as far as practicable is desired to obtain as low cost as may be of wheels and generators. The overgate feature, as previously explained, also makes the high-speed wheel well adapted for low-head installations. Such a wheel is of course poorer in relative efficiency at part gate than the other types, but all things considered the advantages in favor of its use for low-head installations are very decided.

Going to the other extreme, for high-head installations a wheel of the low-specific-speed type is advantageous. A high head tends toward a high actual r.p.m., which may, therefore, be somewhat lessened without loss of economy by using a low-specific-speed wheel. In the high-head development commonly the wheel discharge is relatively small (often water has to be stored), and in general water is more valuable than in the case of the low-head development. Efficiency at part load, therefore, becomes of greater importance, and the low-speed runner is well adapted for these conditions.

While there are no definitely fixed limits of head for which each specific speed is applicable, Moody has developed an empirical formula for specific speed based upon experience, as follows:

$$N_s = \frac{5050}{h + 32} + 19$$

where h is the effective head in feet. This curve was made consistent with a number of runners actually installed which have not shown pitting or cavitation effects and, therefore, gives values of N_s not to be exceeded. It does not apply in the case of propeller-type or impulse wheels.

White has more lately suggested as a limiting value for specific speed the formula

$$N_s = \frac{632}{\sqrt{h}}$$

This gives values of N_s a little larger than Moody's formula, except at high heads.

In Fig. 81 are plotted, on a semilogarithmic scale, values of N_s against h for the wheel units listed in Table 75, Chap. VII, and for various propeller-type wheels. Moody's formula was used to construct the curve so marked, and, as will be noted, the latter gives somewhat higher values of N_s , particularly for heads below 50 to 75 ft. The curve for White's formula lies to the right of the Moody curve, except at very high heads. The plotted points in Fig. 81 for the addition to Table 73, including wheels installed since about 1925, are shown as solid circles. They

indicate a slight tendency toward an increase in N_s for heads above about 50 ft. The propeller-type wheels show a considerable variation in the N_s -head relation. A curve suggested by Karpov¹ is shown for European Kaplan and propeller-type wheels, also numerous points and an approximate curve relation for Nagler propeller-type wheels.

For the highest heads the Pelton wheel is used, also for smaller heads where the available discharge is small. Velocities are very high for these

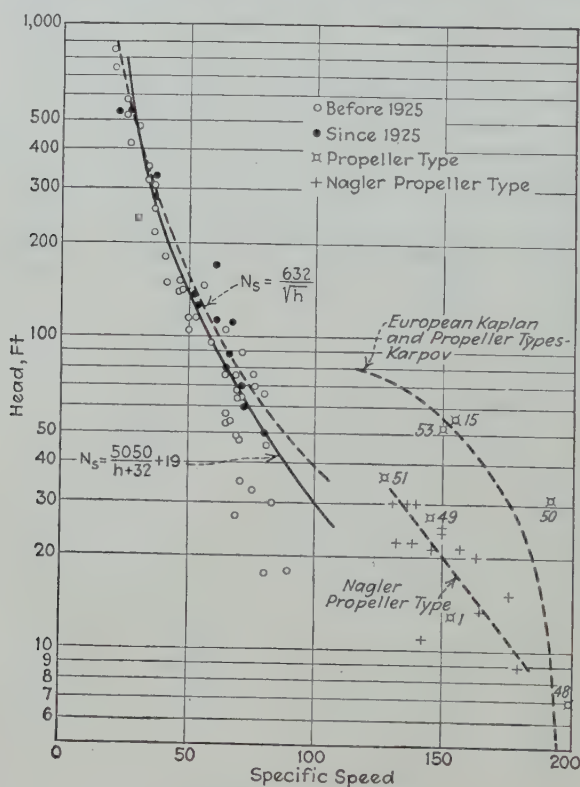


FIG. 81.—Specific-speed head.

wheels, requiring buckets designed for the smallest possible hydraulic losses, with short flow lines. The high velocities result in rapid wearing out of buckets, which, however, are arranged so as to be readily replaced.

At present, specific speeds up to about 4 for a single nozzle are used. By using double-overhung units this may be increased to about 5.5 for the unit as a whole. More than two wheels on a shaft would result in mechanical difficulties. The use of several nozzles is limited by interference of water from the several jets and complication of pipe connections. Any increase in specific speeds of Pelton wheels over the above figures

¹ *Power*, July 15, 1930, p. 109.

is obtained, therefore, only at a sacrifice of efficiency, simplicity, and reliability.

Below heads of about 1000 ft. the Francis wheel and its modification, the mixed-flow type, are used, with specific speeds from 20 to 110. For heads of less than 30 to 50 ft. the propeller-type wheel is now being largely used.

The upper limit in respect to head under which the Francis wheel may be used is due to the following:

1. High velocity of water to and in the runner, resulting in rapid wear.
2. High friction losses due to relatively large runner surfaces, revolving in water.
3. Leakage by clearance spaces becoming relatively large.
4. High pressure making casings difficult to design and build.

For low heads there is no hydraulic limit of size of wheel, but rather an economic limit due to large physical dimensions and low speed. Solid runners are limited to a diameter of about 16 ft. because of transportation. A runner of this size under 15-ft. head, with $N_s = 110$, would deliver only about 3500 hp. at about 55 r.p.m. and would have an outside diameter of about 25 ft., requiring a large and costly power-house superstructure and foundation.

The relation between head and specific speed from the point of view of length of draft tube is discussed further on page 240. Here again to maintain a desirable height of setting above tailrace level, it is necessary to use a wheel of low specific speed for high heads, whereas this height can be readily maintained under low head with wheels of high specific speed.

Use of Propeller-type Wheels.—The propeller type of wheel has extended the field of specific speeds for the single-runner unit from about 110 (the former limit) to about 200. This new type of wheel is particularly adapted for low-head installations and is in wide use for such plants. While its efficiency at the full-load point is somewhat less than that of the medium-speed reaction wheel, the propeller type of wheel is not much lower in peak efficiency than the high-speed mixed-flow wheel. The fixed-blade propeller type, moreover, is very simple in construction and not so likely to become clogged in operating. Its load-efficiency curve has, however, a relatively sharp peak, as already discussed, and a comparatively slight change from the best gate results in a marked lowering of efficiency. This disadvantage has now been overcome by the Smith-Kaplan adjustable-blade wheel, which, as previously noted, has a relatively high efficiency through a wide range of load. While it is somewhat more costly than the fixed-blade wheel, its characteristics may enable a saving in number of units in a plant, thus compensating as to cost.

Propeller-type wheels are limited to an extreme in head of not over about 70 or 80 ft., chiefly on account of cavitation difficulties. Thus far

in this country a head of 60 ft. is the greatest used for this type of wheel. Abroad, the River Shannon development in Ireland will have 33,000-hp. Kaplan wheel units under a head of 106 ft. While cavitation for heads over 70 ft. may be avoided by a setting of wheel below tailrace level, this would usually result in too great construction cost.

Where load conditions are such that the wheels may be readily operated at or close to the best gate, the fixed-blade propeller type of wheel may prove economical as its total weight is considerably less than that of an equal-powered mixed-flow wheel at a given head, and the higher speed of the propeller type results in a generator with somewhat lower cost. For a given capacity and head, the water passages, flume, and draft tube are not essentially different in size from those of the mixed-flow wheel; hence, there is no saving in power house with propeller-type wheels.

The following comparison of weight of wheel units is of interest:¹

Cedar Rapids: $hp = 10,800$, $h = 30$, r.p.m. = 55.6 ($N_s = 82$). Runner $182\frac{3}{4}$ in. in throat diameter, height 11 ft. 9 in., built in four sections of cast iron, each with four vanes, cast-steel crown plate and bottom band. Weight of runner 87.5 tons, of whole wheel 500 tons.

Great Falls—Manitoba Power Co.: $hp = 28,000$, $h = 56$, r.p.m. = 138.5 ($N_s = 151$, propeller type). Runner 189.5 in. in throat diameter, height 4 ft. $5\frac{1}{4}$ in., built in one piece of cast steel. Weight of runner 32.5 tons, of whole wheel 315 tons.

In the Cedar Rapids wheels were used under a 56-ft. head, as at Great Falls, they would give 27,800 hp. at 74 r.p.m. Hence, these two wheel units are directly comparable as giving approximately the same horse power under a given head.

The weight of the propeller-type runner is about 37 per cent, and of the whole wheel 63 per cent of that of the mixed-flow wheel at Cedar Rapids. The r.p.m. of the propeller type is also just about double that of the other wheel.

The basis for determination of the relative economy of the propeller and mixed-flow types is the estimated cost per kilowatt-hour, however, of output in each case, as this takes into account not only first cost of machinery and plant, in the form of fixed charges, but also the output as affected by the wheel efficiency.

Wheel Settings. *Vertical or Horizontal.*—In the earlier water power developments in this country, which were invariably for low heads, varying ordinarily from perhaps 10 to 20 ft., with open wheel-pit installations, it was often the case that the depth of submergence of the wheel below the forebay or canal level had to be slight in order to keep the machinery above the tailrace flood level. Under these conditions the use of vertical settings predominated, owing to the fact that the depth

¹ *Eng. Jour. (Canada)*, September, 1922, p. 461.

of runner was always materially less than its diameter, commonly this ratio of B/D not being more than one-half. Under these conditions for a given depth of water on the center of the wheel, the vertical setting gave materially more submergence.

On the other hand, the vertical setting required beveled gears for connection with a horizontal shaft and pulley used for driving mechanical equipment. Where a horizontal wheel could be used, the necessity of bevel gears was eliminated and such settings were preferred where practicable.

Subsequent to the development of electric power by generators, the use of the horizontal setting was also advantageous, as for many years generators of the horizontal type were the only kind available and adapted for use with the horizontal wheel without gearing.

Multiple-wheel Units with Horizontal Generator.—For very low heads, however, it was often impracticable to obtain a sufficiently high speed without the use of more than one wheel runner. We find, therefore, in the period from about 1900 to 1910 in some cases two, four, six, and even eight or more vertical wheels connected by bevel gears to a single line of horizontal shafting operating one generator. Such an arrangement obviously has serious mechanical difficulties in regard to alignment of bearings as well as operation of the wheel gates.

In some cases two or more horizontal runners were mounted in line on the same horizontal shaft (thus avoiding the use of bevel gears), direct connected with a horizontal generator. With several runners a long wheel pit, shaft, and gate control were required, all under water and relatively inaccessible, resulting usually in poorer care of the wheel and gradual lessening of efficiency.

The development of the vertical type of generator about 1905–1910 and of single-runner wheels of high capacity and speed about 1911 has changed this situation entirely, and the recent development of the propeller types of runners has still further made it possible to utilize low heads at higher speeds.

For illustration, in Fig. 82 is shown a shop view of a line of three pairs 39-in. wheels furnished by the S. Morgan Smith Company for the Green Bay and Mississippi Canal Company, Kaukauna, Wis., about 1909 (?). Two lines of these wheels were furnished, each line to give 1750 hp. (or about 1200-kw. generator capacity) at 180 r.p.m. under a 20-ft. head in an open wheel-pit setting. The method of gate control is clearly shown, as well as the six horizontal bearings and long connecting shaft. The specific speed of each runner was thus about 73, and for the line of wheels as a whole, 178. ($P_u = 0.00214$ for each wheel and 0.0128 for the line of wheels.)

A single-runner wheel unit of similar characteristics to operate a generator of 1200-kw. capacity, or, in other words, to replace one of these

lines of wheels, would thus require a specific speed of 178. For 1750 hp. on the wheel shaft—about 1500-kva. generator capacity, for which a speed of 100 r.p.m. could be used—a specific speed of 99 would be required. This could be approximated with a mixed-flow runner, with P_u = about 0.0045 (see Fig. 72, page 216), for which a 66-in. wheel would suffice. With two vertical wheels in place of the six horizontal runners, using an r.p.m. of 150, a specific speed of 105 would be required. These two wheels would each be about 44 in. in diameter.

A single-propeller-type wheel ($P_u = 0.002 \pm$) running at about 120 r.p.m. could also be used, with $N_s = 120$, a rated diameter of about



FIG. 82.—Three pairs of 39-in. wheels—Kaukauna, Wis.

99 in., a wheel the size of which as a whole would differ but little from the 66-in. mixed-flow, single wheel.

Practically the only field which is now left for the horizontal wheel is in the upper range of heads, usually above perhaps 600 or 700 ft., but occasionally for lesser heads, in cases where the wheel capacity is relatively low and a wheel of relatively small dimensions and small passages is used. In such a case, the more ready accessibility of the horizontal setting may be advantageous. The Marlboro plant (see Figs. 294 and 295) is a good recent example of wheel units of small discharge capacity made horizontal for better accessibility.

Single and Multiple Runners on Vertical Shaft.—As noted above, the use of multiple-runner vertical units geared to long horizontal shafting has been discontinued. During the period 1900–1915 a number of

installations were built of vertical units direct connected to vertical generators but with two or more runners on the vertical shaft, as illustrated by such installations as Sewall Falls, McCall's Ferry, Vernon, etc. Such settings require deep foundations and present difficulties in respect

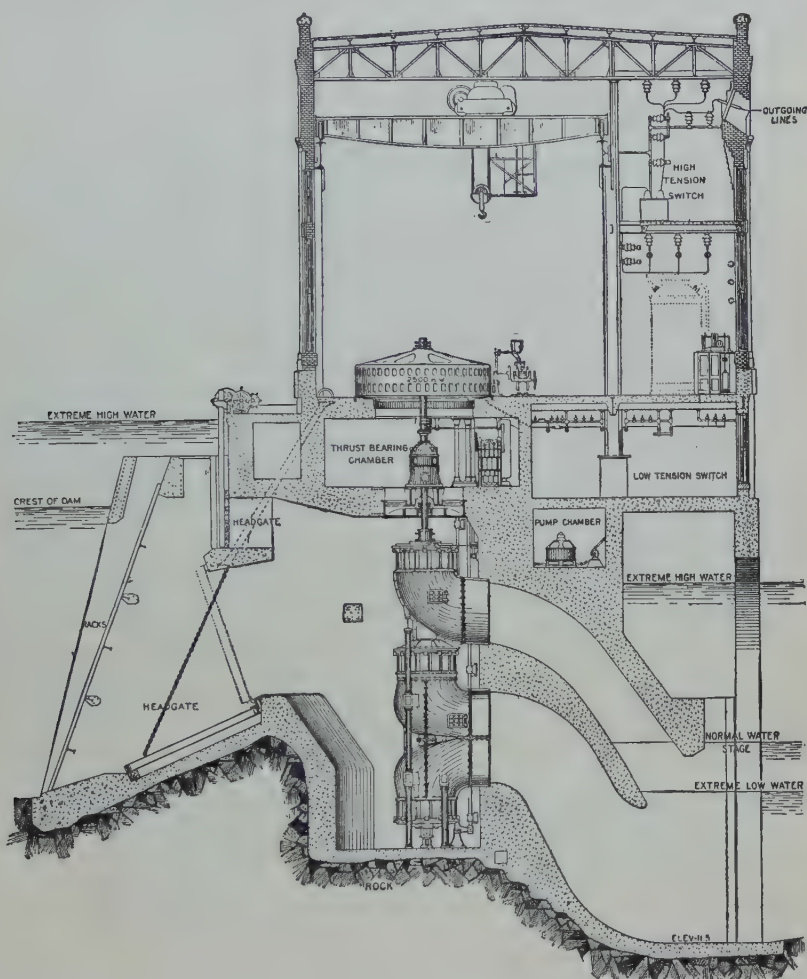


FIG. 83.—Vernon (old) units.

to bearings and wheel-gate operation. They also require more than one draft tube and complicate the entire substructure of the power house.

An interesting setting of this period is that of the Vernon station of the New England Power Company, built in 1909 (see Fig. 83), with triple-runner units, the lower two 60 in. with a common draft tube and the upper runner 57 in. with separate draft tube. These units were rated at 3200 hp. under 130 r.p.m. and a 34-ft. head, with a specific speed of 91

(see Table 57, page 218). The upper single runner was so arranged as to idle on the shaft ordinarily, its particular gate being closed at such times. To increase the power under conditions of backwater, which reduces the head at this plant in the springtime often by 25 or 30 per cent, it was intended to bring into use the single runner. The operation and maintenance of these wheels have been expensive, however, and they are now in process of replacement by single-runner units which will develop about one-third more power at the same r.p.m. under a specific speed of 105 (see Table 57, also Chap. XII).

Experience at Vernon is typical in regard to multiple-runner vertical units, and their use is now obsolete. As already stated, single-runner vertical units predominate almost exclusively.

Type of Wheel Casing as Determined by Head.—The open-flume type of setting (a simple rectangular pit in which the wheel is set) is now in use up to heads of about 20 ft. For the range of heads between about 20 and 60 ft. the best practice is now quite general in the use of the concrete scroll or spiral flume with outside operating gate mechanism for the wheel. The requisite dimensions of such flumes in this type of setting fixes the spacing of the units rather than the size of generators. Common arrangements of the concrete scroll flumes are shown in Figs. 166 and 168, Chap. VII, the purpose being to keep the dimensions of the spiral as small as possible and, therefore, minimize the unit spacing and also gradually to accelerate the velocity of the entering water.

Above a head of about 60 ft. a steel scroll casing is commonly used, although reinforced concrete has been used in a few cases for somewhat higher heads. Similarly, plants with steel scroll casings have been built for heads lower than 60 ft., or, in other words, these types overlap somewhat. Plate steel has been commonly used for the wheel casing up to a head of about 350 ft., an example of such an installation being at the Davis Bridge (Harriman) plant of the New England Power Company (see Chap. XIII). This type of wheel casing is really an extension of the steel penstock around the wheel.

Above heads of about 350 ft. to about 850 ft., cast-steel scroll casings are used. For very large scrolls the difficulties of casting have been obviated by casting the speed-vane ribs integral with the casing and dividing this into a number of sections. Above heads of about 850 ft., as previously noted, the impulse or Pelton wheel is used.

The foregoing represents the best modern practice in respect to wheel settings. Many installations of wheels with cylindrical casings have been used and are still made for moderate heads using horizontal units, as this type of wheel has a smaller first cost than that with the scroll casing; the horizontal generator is also less expensive than the vertical. To offset the lower first cost, however, is a materially lower wheel efficiency due to irregularity in the area of water passages from the penstock

to the wheel runners. Such units have been, in some cases, connected at one end with the penstock, resulting in a setting in which it is difficult to make accessible the bearing at the penstock end. In other cases the penstock has entered the wheel casing from one side or from above. While under certain conditions such installations may still be used, the use of the scroll-steel or concrete flume is now decidedly predominant.

Other types of wheel casings that have been used to a limited extent in the past are conical, globular, and spherical casings.

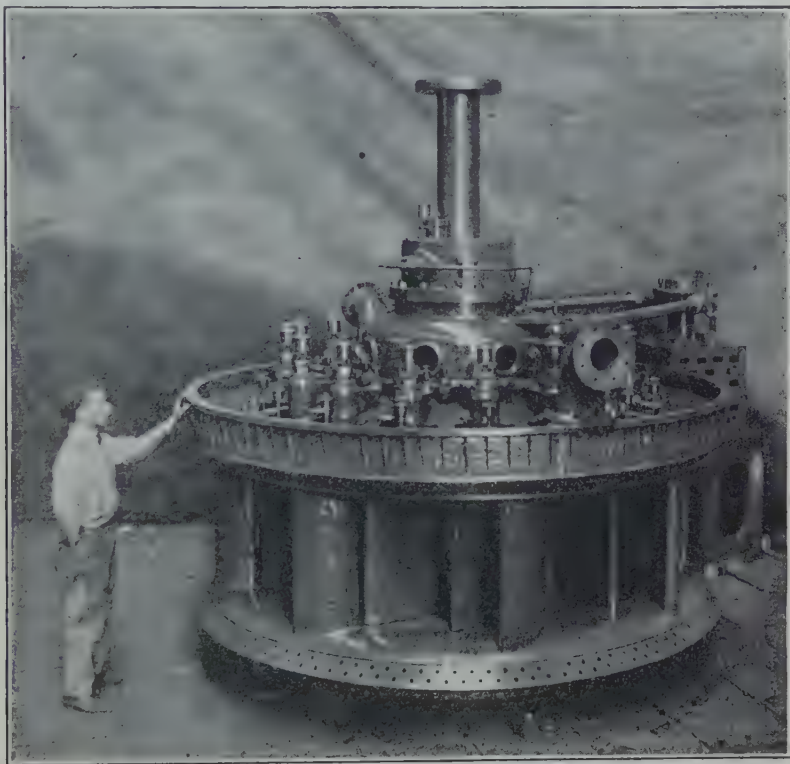


FIG. 84.—Shop view of wheel showing wicket gates and controls. (Courtesy of Newport News Shipbuilding and Dry Dock Company.)

Wheel Gates.—The simplest form of wheel gate is what is known as the “cylinder” gate, which is a thin hollow cylinder so arranged as to move over the guide openings and control the entrance of water to the wheel. Such a wheel gate is very inefficient at any gate other than full gate, owing to the contraction of water, followed by sudden expansion, in passing the gate. Where a plant can be run practically all the time at full gate, such a wheel gate is satisfactory, but with the usual hydro-electric plant, which must of necessity operate much of the time with units at part gate, this type of gate is not practicable.

The modern type of gate in general use is the wicket gate, which is really a combination of gates and guides consisting of vanes rotating above pivots which are fastened to a shifting ring by a link motion, this shifting ring being operated by pressure cylinders actuated by the governor. This type of gate can be thus externally controlled and may be used for either horizontal or vertical single-runner settings. Figure 84 is a shop view of a Newport News Company 3000-hp., 65-ft. head, 277-r.p.m. wheel recently installed at the Locks Power station of the Virginia Railway and Power Company, Petersburg, Va. This clearly shows the speed ring, wicket gates and guides, and gate-shifting mechanism.

As has been noted (see page 223), the wicket gate is designed to give the proper entrance velocity and direction of flow of water to the wheel to fit usually the full-load point. At other wheel gates, as commonly arranged, owing to change in entrance angle and water velocity, shock losses are caused at entrance to the wheel runner. The recent development of Kaplan propeller-type wheels to obviate this situation and to better the wheel efficiency at part gate consists in turning the vanes themselves so that, as the wheel gate is opened, the vane is always kept at the angle suitable to the direction of the entering water. Thus shock losses are minimized and better efficiency at part gate is maintained.

Bearings.—Bearings for vertical wheel units are of two kinds: (1) shaft bearing intended simply to keep the runner and its shaft central with surrounding guides and casing, usually of lignum vitae with water lubrication or of the babbitted type with oil lubrication and placed on the shaft just above the runner (see Fig. 55, page 190); and (2) a thrust bearing located on top of the generator-stator frame and carrying the weight of the rotating parts of wheel and generator as well as the runner thrust.

A recent development in shaft bearings consists of the use of rubber-guide bearings with water lubrication. The first installation of this kind was at the Holtwood, Pa., plant in 1924, since which time rubber bearings have been successfully used in over 30 plants. It is found that the resilient characteristics of the rubber prevent wear of the bearing from grit and sand in the water, whereas the lignum-vitae bearing wears rapidly if the water contains abrasive material.

The development of suitable types of thrust bearings has been an important adjunct in the use of vertical units. The types now commonly in use consist of the Kingsbury, Gibbs, and the spring thrust bearing.

The Kingsbury thrust bearing¹ (Kingsbury Machine Works) is a contact bearing running in an oil bath with the weight carried on a set of segmental, babbitted shoes, with space between shoes for oil circulation. Each shoe has a single pivot support a little beyond the center of gravity

¹ *Eng. Record*, Jan. 11 and 18, 1913.

in the direction of rotation, causing a very slight tilting of the shoe with reference to the runner thrust block, which tends constantly to draw in the oil as the runner turns.

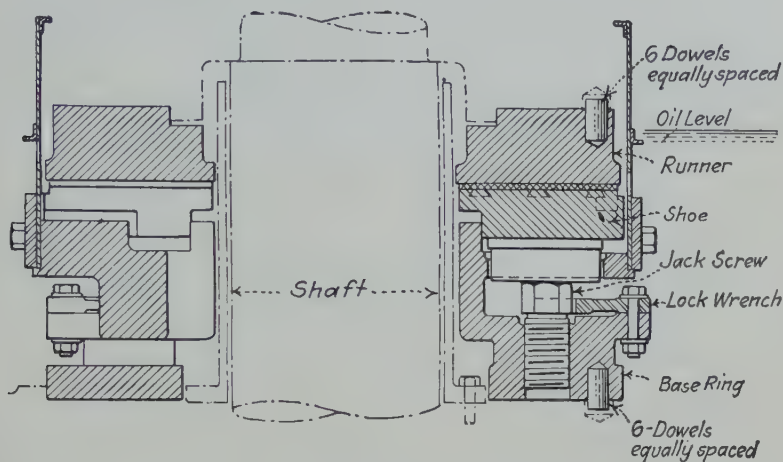


FIG. 85.—Kingsbury thrust bearing.

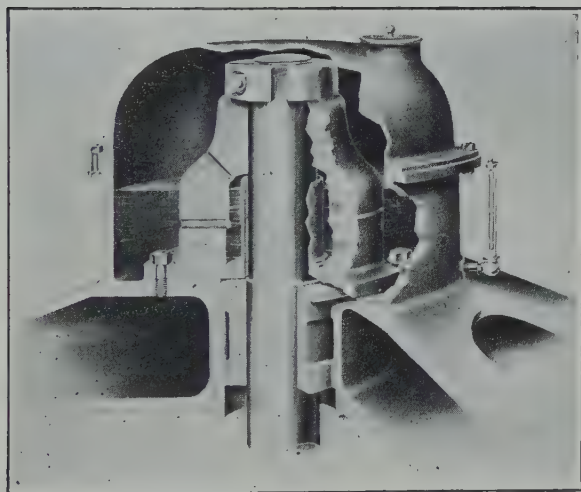


FIG. 86.—Gibbs thrust bearing, sectional view.

In Fig. 85 is shown a sectional view of one type of Kingsbury bearing for large units, arranged with six segmental shoes and made in capacities from 69,000 to 261,000 lb. when carrying 350 lb. per square inch.¹ The safe limit for these bearings is 500 lb. per square inch.

The Gibbs thrust bearing (S. Morgan Smith Company) (see Fig. 86) has an upper rotor ring attached to the shaft and turning with it. The

¹ *Bull. E*, Kingsbury Machine Works, p. 5.

(lower) stator ring is attached to the stator frame and is stationary. The stator ring is divided into segmental faces (usually eight) by radial grooves. The segmental faces are very slightly beveled away from the grooves in a tangential direction, rising in the direction of rotation to a point about midway on the segmental face. As the wheel turns, a film of oil is constantly pulled in between the two rings.

The spring thrust bearing (General Electric Company), like the Gibbs bearing, has the rotor and stator rings. The latter is, however, supported

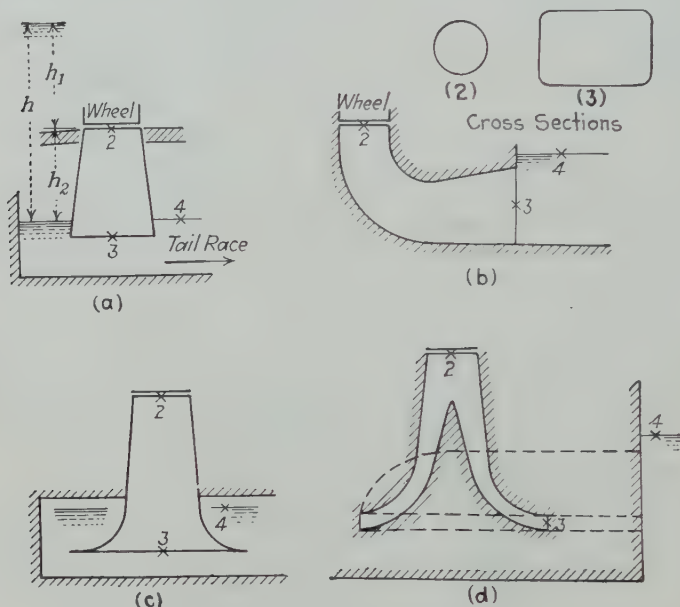


FIG. 87.—Types of draft tubes.

by nests of heavy springs, thus giving a flexible support which will automatically adjust itself to running conditions.

Draft Tubes.—The draft tube is an extension of the wheel passages from the point of exit from wheel runner down to tailrace level. Figure 87, *a* to *d*, shows diagrammatically some different forms of draft tubes. It is analogous to having the wheel discharge into a vertical pipe, usually flaring or increasing in diameter so as to lessen the velocity at exit from the tube. This exit velocity head thus becomes the exit loss rather than the velocity head at exit from the wheel runner (or entrance to draft tube). To have a downstream or horizontal flow away from the draft tube, it is often curved as shown in Fig. 87*b*.

The primary purpose of the draft tube is to permit the placing of wheel and connecting machinery at a level above that of water in the tailrace under high-water or flood conditions of the river. Its secondary

purpose is to reduce velocity-head losses of water leaving the wheel, as previously explained.

The diffuser as used on the old Boyden wheel accomplished the secondary purpose of a draft tube, but the wheel was set below tailrace level and the diffuser thus simply lessened exit velocity from the wheel.

Theory of Draft Tubes.—Referring to Fig. 87a, the following equation may be written as between points (2), at entrance to draft tube, and point (4), at tailrace level (using gage pressures):

$$\frac{V_2^2}{2g} + \frac{p_2}{w} + h_2 = \frac{V_4^2}{2g} + 0 + 0 + h_f + \frac{V_3^2}{2g}$$

where h_f represents friction losses in the draft tube. Transposing and noting that $V_4^2/2g$ is usually small and may be neglected, we have

$$\frac{p_2}{w} = -h_2 - \frac{V_2^2}{2g} + h_f + \frac{V_3^2}{2g} \quad (17)$$

$$\text{and} \quad h_2 = -\frac{p_2}{w} - \frac{V_2^2}{2g} + h_f + \frac{V_3^2}{2g} \quad (18)$$

From this approximate analysis it is evident that for a given height of draft tube there is a limiting value of V_2 , or, conversely, a given value of V_2 limits the height of draft tube.

Practically it will usually be desirable to limit the height of draft tube to between 15 and 20 ft. and, hence, of V_2 to between about 20 and 25 ft. per second, and in general h_2 must be less as the head on the wheel increases.

For example, a high-specific-speed wheel ($N_s =$ about 90) under a head of 50 ft. would have a velocity (V_2) at exit from runner of between 17 and 20 ft. per second or $V_2^2/2g =$ about 5 ft. Tube friction loss and exit-velocity head ($V_3^2/2g$) would not exceed 1 ft., so that

$$h_2 = -(-25) - 5 + 1 = 21 \text{ ft.}$$

Under a 300-ft. head this same runner would have

$$V_2 = 18 \times \sqrt{\frac{300}{60}} = 40$$

$$\text{and} \quad \frac{V_2^2}{2g} = 25$$

$h_f + \frac{V_3^2}{2g}$ would be about 5 ft. Then

$$h_2 = -(-25) - 25 + 5 = 5 \text{ ft.}$$

Or, as the head increases, the wheel would have to be lowered with reference to tailwater level.

Hence, as the head increases, there would result a situation where the wheel would have to be at tailrace level if the same type of runner

continues to be used. For higher head, therefore, it becomes necessary to change the type of wheel and lower the value of $V_2^2/2g$.

Thus in the above example, if a runner is used with $N_s = 36$, which is adapted for a head of 300 ft., its V_2 would be about 23 ft. per second and $V_2^2/2g =$ about 8, under which condition

$$h_2 = -(-25) - 8 + (1+) = 18 \text{ ft.}$$

or about the same permissible height above tailwater level as with the low-head setting of 50 ft.

An inspection of Eq. (17) will indicate that with ordinary values of its terms p_2/w , or the pressure head at exit from wheel runner (or entrance to draft tube), will be negative or less than atmospheric (as h_f is relatively small and $V_3^2/2g$ always less than $V^2/2g$ with a flaring draft tube, so that the last three terms in Eq. (17) will either balance (or have some negative value). The theoretical limit in p_2/w is of course -34 ft. (gage pressure), but practically it must be kept at or below -25 ft. Hence, in Eq. (18) h_2 , or the height of wheel above tailrace, is limited to about 25 ft. as an extreme.

Also referring to Eq. (18), it will be seen that for a given value of h_2 , assuming $h_f + V_3^2/2g = 1$ approximately and $p_2/w = -25$, $V_2^2/2g$ and V_2 will be limited as follows for different values of h_2 :

Assumed height of draft tube, h_2	Maximum allowable values	
	$\frac{V_2^2}{2g}$	V_2
25	1	8
20	6	20
15	11	27
10	16	32

The relation of head and specific speed in selecting suitable types of wheels is thus emphasized from another point of view, and it is seen that in general the proper value of N_s varies inversely with the head.

An interesting phase of this matter is the suggestion by Taylor of inverting the wheel, admitting the water below it and discharging it vertically upward, thus permitting the wheel to be set a considerable distance below tailrace level, and, hence, the use of a higher-specific-speed wheel. Discharge would be over a circular weir and siphonic.

Factors Affecting Dimensions of Draft Tubes.—The dimensions of draft tube at exit from wheel runner are practically fixed by the wheel dimensions, the velocity V_2 varying from $0.4\sqrt{2gh}$ with high values of N_s (low heads) to $0.10\sqrt{2gh}$ with low values of N_s (high heads). The distance above the tailrace level is fixed within limits by the principles

just described, also keeping in mind as far as necessary the desirable height of wheel above tailrace level to avoid flooding of machinery or power house at high-water stages. The desirable velocity and velocity head at exit from draft tube are also practically fixed within limits by the

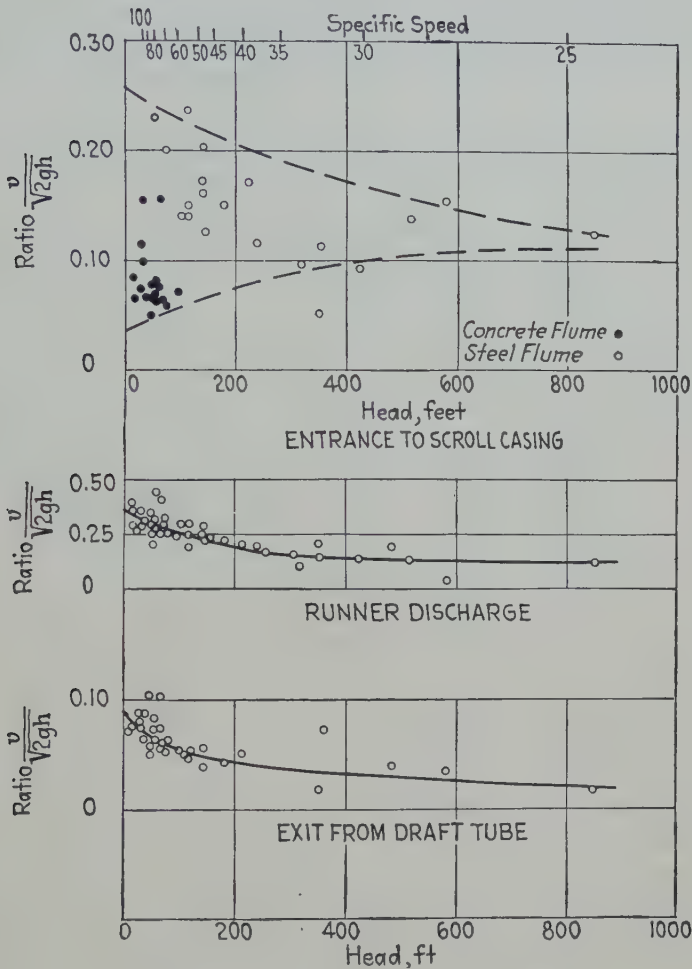


FIG. 88.—Velocity ratios for reaction wheels.

type of wheel or head, V_s being usually from 0.10 to $0.02\sqrt{2gh}$, with the large values for higher values of N_s (lower heads).

In Fig. 88 are shown plotted against head (and auxiliary scale of N_s) velocity ratios at three points with reference to $\sqrt{2gh}$, viz.: (1) at exit from draft tube, (2) at discharge from runner (or entrance to draft tube), (3) at entrance to the scroll casing. The wheels included are those listed in Table 75, Chap. VII. Velocity ratios are greatest for low heads (high N_s) and vary from:

1. At draft-tube exit . . . 0.10 to 0.02.
2. At runner discharge . . 0.40 to 0.15.
3. At entrance to scroll casing . . . no well-defined curve, average about 0.13; concrete scroll cases about 0.07.

The flare of the tube as indicated from experience in operation, particularly at part gates, should not usually exceed about 3 in. per foot of length as a maximum value and should be less where conditions permit this.

The following table based upon the curves in Fig. 88 will be found useful in determining limitations in height of wheel above tailrace level.

TABLE 58.—VELOCITIES AND VELOCITY HEADS AT RUNNER DISCHARGE AND DRAFT-TUBE EXIT—BASED UPON FIG. 88

Head, feet	$N_s = \frac{632}{\sqrt{h}}$	$\sqrt{2gh}$	$\frac{V_2}{\sqrt{2gh}}$	V_2	$\frac{V_2^2}{2g}$	$\frac{V_3}{\sqrt{2gh}}$	V_3	$\frac{V_3^2}{2g}$
30	114	44	0.34	15.0	3.5	0.07	3.1	0.2
50	89	57	0.30	17.1	4.5	0.065	3.7	0.2
100	63	80	0.25	20.5	6.5	0.055	4.4	0.3
200	45	114	0.19	21.4	7.0	0.045	5.1	0.4
300	36	139	0.16	22.7	8.0	0.035	4.9	0.4
400	32	161	0.15	24.0	9.0	0.03	4.8	0.4
600	26	197	0.13	26.0	10.5	0.025	5.0	0.4
800	22	228	0.12	28.0	12.0	0.02	4.5	0.4

It should be noted that the above values are based upon the mean curves shown in Fig. 88. Individual wheels show considerable departures in both directions from the figures in this table.

To indicate some of the elements affecting the design of draft tubes in Table 59 are given certain factors of velocity, area, etc., for the draft tube of an assumed 2000-hp. wheel unit operating under heads of 11, 55, and 275 ft., respectively, and (on the basis of 80 per cent wheel efficiency) requiring discharges of 2000, 400, and 80 sec.-ft., respectively. Velocities are consistent with the curves in Fig. 88 and values in Table 58.

Draft-tube areas A_2 at entrance and A_3 at exit (11) or (12) are consistent with velocities V_2 and V_3 , respectively. The ratio of these areas (13), as well as approximate diameter or cross-dimensions (14) or (15), indicates the relative length of draft tube to obtain any given flare. These factors, together with the necessary length of tube in order not to exceed a flare of 3 in. per foot (16), show clearly the greater relative difficulty of the low-head setting. In the case assumed for a head of 11 ft., the resulting length and size of tube are so large as to be impracticable, and a greater value of V_3 and consequent greater exit losses will have to be assumed. As a matter of fact, the value of N for the unit would be

about 50 r.p.m., which would be undesirably low for a 1400-kw. generator (to fit these wheel units) and would suggest the use of smaller units.

For the 275-ft. head the required dimensions of draft tube are small owing to the relatively small value of Q under this head, and the tube could be made longer and with larger end area. The tendency is, therefore, to assume a somewhat greater percentage of outflow losses with low-head settings and a smaller percentage with high heads.

TABLE 59.—DRAFT TUBES—COMPARISON OF REQUIRED VELOCITIES, AREAS, ETC., WITH VARYING HEAD

Num- ber	Item	$h = 11$ $\sqrt{2gh} = 26.8$ ($N_s = 110$)	$h = 55$ $\sqrt{2gh} = 59.5$ ($N_s = 60$)	$h = 275$ $\sqrt{2gh} = 133$ ($N_s = 35$)
1	Horse power	2000	2000	2000
2	Q	2000	400	80
3	$\frac{V_s^2}{2g}$	0.09	0.27	0.42
4	V_s	2.4	4.2	5.2
5	C in $V_s = C\sqrt{2gh}$	0.09	0.07	0.04
6	Horse power lost at exit from draft tube	16	10	3
7	$\frac{V_s^2}{2g}$ (per cent of h)	14	9	2.5
8	$\frac{V_s^2}{2g}$ in feet	1.5	5.0	6.9
9	V_s	9.9	18.0	21.1
10	C in $V_s = C\sqrt{2gh}$	0.37	0.30	0.16
11	A_2	202	22	3.8
12	A_3	835	95	15.3
13	Ratio A_3/A_2	4.1	4.3	4.0
14	D_2 (circle)	16.0	5.3	2.2
15	$B_3 \times D_3$ (rectangles, $B_3 = 2D_3$)	41.0 $\times$ 20.5	13.8 $\times$ 6.9	5.6 $\times$ 2.8
16	Length required for flare of 3 in. per foot in width, B_3	100	34	13.6

The proper amount to allow for outflow losses is a matter of economy. Theoretically, as with the penstock line, the annual value of power lost plus cost of tube (or features affected by it) should be a minimum. Practically, the cost of draft tube or its effects is difficult to segregate and the value of the power lost also uncertain, so that reasonable values of outflow velocity based upon judgment and experience must be adopted.

It is evident from Table 59 that the spacing of wheel units for low- and medium-head plants is likely to be affected or fixed by the necessary dimensions of the draft tube and scroll case, while for higher heads this will not be the case and clearance required between generators will usually dominate.

Types of Draft Tubes.—The common flaring vertical draft tube (Fig. 87a) is usually made of steel plate and often furnished as part of the wheel equipment. Occasionally, to give exit velocity a downstream direction, the tube is curved in its upper portion so that it discharges at about 45 deg. with the vertical.

A long straight vertical tube, with small angle of flare, is probably most efficient in reducing exit losses, but with large units this would mean excessive excavation and substructure cost; hence, the use of a curved or quarter-turn tube. The quarter-turn draft tube (Fig. 87b) is commonly molded in the concrete substructure of the power station and, as shown, is flattened toward exit to decrease the amount of excavation and substructure. Instead of the rectangle with curved corners as shown, the section is often made elliptical.

The hydraucone draft tube (Fig. 87c) patented in 1917 by W. M. White, chief hydraulic engineer of the Allis-Chalmers Company, utilizes the so-called "hydraucone action" of a jet of water impinging on a plate (usually flat, but may be conical, convex, or concave, but preferably concentric with the axis of the entering stream) and being deflected therefrom in all directions. It may be constructed of steel plate as shown or molded in the concrete substructure; it permits the out flowing water to change in direction of flow through 90 deg. and pass downstream and at the same time requires less depth of substructure than either the vertical or quarter-turn tube.

The Moody or spreading draft tube (Fig. 87d) obtains substantially the same effect as the hydraucone by an actual cone (usually of concrete, with steel tip) constructed symmetrically in the tube. The form of the Moody tube and cone is based, however, upon that of an inverted "free vortex," similar to the familiar form which occurs in discharge through the outlet in the bottom of an open basin.

The hydraucone and Moody draft tubes also permit of the better utilization of the "whirl component" of water entering the draft tube, a matter of especial importance with high-speed wheels under low heads. As water enters the draft tube, it is whirling in the direction in which the runner rotates, the center of the vortex being the vertical axis of the draft tube. Due to its gyroscopic properties, this vortex tends to remain in the same plane and will not, therefore, follow the center line of the quarter-turn tube. This results in serious eddies and whirls in the curved portion of such tubes, and at its exit only a portion of the discharge area may be effective. Water may actually flow back into part of the tube, thus causing serious eddy losses and high outflow losses in the other portions of the tube. The hydraucone and Moody draft tubes are symmetrical about the axis of whirl, thus minimizing eddy losses and reducing the whirl component to a small value at exit from the tube.

Progress in Draft-tube Design.—The development of the best form of draft tube, particularly with the lower-head installations where wheels of high specific speed are used, with relatively high velocities at entrance to draft tube, has been given much study. Recent experimental investigations include:

1. The hydraucone regainer, its development, and application in hydroelectric plants (W. M. White, *Jour. A.S.M.E.*, May, 1921).

2. Tests of five model draft tubes at Muscle Shoals (*Eng. News-Record*, Aug. 2, 1923, page 182).

3. Comparative tests of experimental draft tubes, made at the Worcester Polytechnic Institute hydraulic laboratory for the Alabama Power Company (*Trans. A.S.C.E.*, vol. 87, page 1547, 1924).

The more recent tests of model draft tubes at Muscle Shoals and Worcester show a slight advantage in efficiency with the Moody or spreading draft tube, with the high cone extending to the top of the tube. This type of construction now prevails with the more recent developments of this kind, particularly where propeller-type wheels are used. The hydraucone tube shows but slightly less efficiency, however, and is less expensive in construction cost. Both forms of tube are in very extensive use in the recent installations, the horse power developed with hydraucone draft tubes being somewhat less than that with the spreading type, but covering a greater range in size of units and head utilized.

A recent interesting installation of hydraucone and also of quarter-bend and flat-plate type at the Mitchell dam of the Alabama Power Company utilizes a form of "backwater suppressor" or "fall increaser" consisting of an extended horizontal discharge channel so arranged as to be exposed at its downstream end to high velocity of flow from water taken over a spillway passage under the power house, thus maintaining an approximately normal head at times of high water.

The arrangement and design of draft tubes are further discussed in Chap. VII.

Cavitation Effects or Pitting of Turbine Runners.¹—Pitting of turbine runners has occurred in all types of runners and kinds of metals. In atmospheric corrosion the products of corrosion remain on the surface, while with a pitting runner the products are carried away and the pitting surface exposed to further attack, resulting in a very quick process. In severe cases, runner buckets 2 in. thick have been eaten through in less than 2 years.

Nearly all of the pitting of turbine runners has occurred on the back or vacuum side of the wheel, suggesting that the degree of vacuum or the draft head acting on the runner is a factor in pitting.

¹ "Pitting of Hydraulic Turbine Runners," *Report Hydraulic Power Committee, Nat. Electric Light Assoc.* pp. 820-851, 1925.

The results of an investigation of some 250 turbines in operation under a wide range of conditions has confirmed this, as will be noted in Table 60, which shows clearly that pitting increases with draft head.¹ It undoubtedly occurs where the main stream of water pulls away from the bucket

TABLE 60.—EFFECT OF AVERAGE TOTAL DRAFT HEAD ON PITTING

Diameter of wheel, inches	Average total draft head, feet		
	A No pitting	B Moderate pitting	C Excessive pitting
Up to 48	19.0	24.9	27.5
48 to 96	16.2	23.9	25.9
Over 96	19.0	20.6	22.7

surface and may be due to poor runner design, such as sharp curves, humps, or depressions in the bucket surface. However, draft head is of greatest importance and a poorly designed wheel may be set with a low height above tailwater and show no pitting, while a properly designed wheel may be set so high above tailwater that pitting is certain to occur.

Causes of Pitting.—Pitting of turbine runners is due to the effects of cavitation, which occurs in flowing water when a cavity or void filled with air and water vapor is formed. When the pressure at a given point drops to a certain value, the coherence of the water is broken and air with its constituent elements, oxygen, nitrogen, etc., is freed from the water and a bubble or series of bubbles formed, which further aid in vaporizing the water. With low pressure the quantity of air is small compared with that of water vapor.

In the case of the hydraulic turbine these small voids or bubbles occur in eddies or vortices, and, as these pass over the casing or runner and get into a higher pressure, they suddenly collapse. The impact of the collapse compresses the air bubble, and extremely high pressures (approximated at 1000 atmospheres) result. Furthermore, the small bubble of air so compressed enters the small crevices or clefts in the material, acting like an explosive and causing a rapid breaking up of the material. The action due to cavitation is therefore fundamentally mechanical, although some minor chemical effects may occur. This "hammering" acts not only normal to the surface, but there is also a tangential effect which breaks down the material very rapidly because the bubble is moving together with the flow when it collapses.

These effects cause great turbulence, aggravated by the formation of cavities. The resulting destruction appears to be less with more elastic

¹ In this table "draft head" is the sum of the vertical distance between runner exit and tailrace level plus velocity head at runner exit.

materials. Brittle materials like glass show a more marked effect than bronze. It also appears likely that an increase in temperature occurs, which would explain the curious appearance—like melted lava—of the damaged surface.

Further experimental study of cavitation and its effects is now under way at the Massachusetts Institute of Technology.

Operating Suggestions.—From an operating standpoint higher speed, higher head, or greater capacity; lowering of tailwater or improvement of draft-tube efficiency; and finally, especially under high heads, operating at gate openings other than best gate, all contribute toward pitting. Many cases of pitting have occurred where wheels have been overgated to secure an increase of power. It is not confined to any one class or type of turbine nor does it depend upon the kind of metals used.

Pitting may be avoided by a lower wheel setting or any other method which reduces the draft head. Draft head has in some cases been reduced by admitting air into the draft tube, but this reduces the efficiency of the wheel. Draft head may also be reduced by carrying the load at or close to the best gate.

The important factors influencing pitting are then:

1. Height of runner above tailwater.
2. Draft due to average velocity at runner throat.
3. Runner design as to bucket size, angle, and curvature as affecting tendency of the stream to "pull away."
4. Draft-tube design.

References

1. SPANNHAKE, WILHELM, "Cavitation and Its Influence on Hydraulic Turbine Design," Nat. Electric Light Assoc., *Pub.* 222, June, 1932.
2. ENGLESSON, "Pitting in Water Turbines," *The Engineer*, Oct. 17, 1930.
3. KARPOV, A., "Low-head Hydroelectric Developments," *Trans. A.I.E.E.*, vol. 52, p. 202.
4. KEMPF and FOERSTER, "Hydromechanische Probleme des Schiffsantriebs."

TURBINE TESTS

Tests of hydraulic turbines may be:

1. Holyoke tests, so called, made at the testing flume of the Holyoke Water Power Company, Holyoke, Mass., the only general commercial wheel-testing flume at present. (Note that wheels may also be tested to a limited extent as to capacity at the various hydraulic laboratories of technical institutions in this country and at laboratories maintained by several of the larger wheel manufacturers.)

2. Tests in place, usually for the purpose of acceptance tests or after a plant has been in operation for some time, to see whether wheel efficiency is being maintained.

Holyoke tests are usually complete tests covering a range of all ordinary wheel-gate openings and also of different speeds for each gate, thus furnishing data for complete characteristic curves of the wheel.

Tests in place are usually for different gate openings but constant r.p.m. in accordance with generator speed but if possible with some variation in head for different runs, especially in the case of a low-head installation, subject to occasional backwater effect.

Essentials of Turbine Tests.—The essential result desired from a turbine test is the wheel efficiency at different gates (as well as different speeds in the Holyoke or complete test), which is the ratio of output to input of power expressed in percentage form.

The elements entering into these two factors which will now be considered are as follows:

A. Input:

1. Head.
2. Discharge.

B. Output (as measured by dynamometer):

1. Load or weight on dynamometer.
2. Effective diameter and lever ratio of dynamometer.
3. Speed in r.p.m.

The formulas for power input or hp., [Eq. (2), page 208] and power output hp. [Eq. (8), page 211] have been previously given and require no further discussion, except to note that in Eq. (8) D is the effective diameter of the dynamometer in feet and P the effective weight or load (the actual weight $\times$ the lever ratio of the dynamometer).

A. 1. *Head.*—This is the effective head on the wheel as noted on page 209 [Eq. (4)] or the total head above tailrace level available at entrance to wheel runner. For an open flume setting this will be the difference in level of water in flume above and in tailrace below the wheel. For the concrete scroll-flume setting often the water level at entrance to the flume (inside racks and gates) would be used, although this would be charging to the wheel the losses of head in the flume. If desired, this may be avoided by piezometer or pressure-gage readings at entrance to the wheel itself. With a penstock setting, gage readings must be obtained just above the wheel.

To obtain an accuracy of 1 per cent in head measurements as affecting power input (which varies as $h^{3/2}$), results should be as follows:

Head, Feet	Allowable Error in Measuring
	Head, Feet
10	0.07
100	0.70
500	3.5
1000	7.0

Keeping in mind the limit in accuracy of determining power input imposed by measurement of discharge, which, as will be further noted, is difficult or impossible to determine within 1 per cent, it is seen that only for very low heads need the head be measured nearer than 0.1 ft. to give consistent accuracy, and for higher heads the nearest foot or even pound per square inch on the pressure gage will be sufficient.

A. 2. Discharge.—This is the factor usually most difficult and expensive to determine in wheel tests. Methods in use for the measurement of discharge consist of the following: (a) weir, (b) current meter, (c) Pitot tube or Pitometer, (d) salt or chemical dilution, (e) salt velocity, (f) pressure variation (Gibson), and (g) other methods.

a. The use of a weir presents difficulties of cost as well as interference with normal operating head, as some head must be utilized by the weir itself, whether placed in canal or tailrace. Measurements by standard weir are fairly accurate and dependable, however, and in some cases tests of wheels in place have been made in this way. It is used in measuring discharge at the Holyoke testing flume and is generally recognized as a standard method.

b. Current meter measurements are usually relatively low in cost and may be satisfactory if suitable conditions of velocity and smooth flow prevail, in either canal or tailrace. Even under the best conditions, however, an accuracy of 2 or 3 per cent is difficult to obtain, and often the absence of a measuring section free from turbulence and boiling will prevent the use of current meter. The Fteley or propeller type of current meter is usually best adapted for conditions likely to prevail in testing wheels, the Price meter tending to overregister with turbulent flow. For velocity determinations the $\frac{2}{10}$ - and $\frac{8}{10}$ -depth method, supplemented by numerous vertical velocity curves, will usually be most suitable.¹

c. The Pitot tube or Pitometer is often of service in penstock installations where good velocities prevail. For its use a stuffing-box connection with penstock must be made, and usually traverses with velocity determinations at numerous points must be made on two diameters at right angles with each other. This method gives fairly good accuracy with good velocities and is also relatively low in cost. It can be used, however, only in penstock installations.²

d. In the salt or chemical-dilution method, a salt solution of known strength is injected at a known rate into the water above the wheel, say at the racks. The salt content (if any) of water coming to the racks

¹ See Liddell, "Stream Gaging," for further details of current meter measurements and methods to be used.

² For theory and use of Pitot tubes see Russell, "Hydraulics," pp. 126-132, 1925; also Hughes and Safford, "Hydraulics," Chap. VII, 1911; or King and Wisler, "Hydraulics," pp. 65-69, 1922.

and of water in the tailrace as affected by the injected salt solution is determined, from which the degree of dilution of the salt solution and rate of discharge through the wheel may be determined. Thus, if points 1, 2, and 3 are above the racks, at the racks where the salt solution is injected and in the tailrace, respectively, and further,

W_w = weight of water per second passing the wheel,

W_s = weight of salt solution of known strength injected at the racks per second,

r_1, r_2 , and r_3 = ratio of salt to water at points 1, 2, and 3, respectively, then pounds of salt per second which was injected must be the same at points 2 and 3, or

$$W_s r_2 = W_w (r_3 - r_1)$$

In this equation W_w is the only unknown and may be determined. The values of r_1, r_2 , and r_3 may be determined chemically or indirectly by measurement of the electrical resistance of the solution.

The salt-dilution method gives accurate results but requires an elaborate arrangement of pipes, etc., for injecting the salt and, for large wheel units, requires large quantities of salt and is an expensive method for determining discharge. Its especial field of use is in the case of large wheel units without penstocks, where facilities for good current meter measurements may be lacking.

A very detailed and comprehensive article on the use of the salt-dilution method in testing turbines was presented by B. F. Groat in 1915.¹

e. The salt-velocity method as devised by Prof. C. M. Allen² is based upon the fact that salt in solution increases the electrical conductivity of water. Salt solution is introduced near the upper end of the conduit, and the passage of the solution across one or more pairs of electrodes at other points in the conduit is recorded graphically by electrical recording instruments. The passage of the salt solution between two points is accurately timed, and the volume of the penstock between the same points is accurately determined. The discharge in cubic feet per second equals the volume in cubic feet divided by the time in seconds.

As will be seen, this method is analogous to float measurements made simultaneously by many floats, in that the volume of the prism of water passing between the initial section and the electrodes in a measured time is obtained.

During the fall of 1922 Professor Allen made 10 wheel tests by this method, including steel penstocks from 11 to 13 ft. in diameter, 500 to

¹ GROAT, B. F., "Chemihydrometry and Precise Turbine Testing," *Trans. A.S.C.E.*, vol. 80, p. 951, 1915.

² ALLEN, C. M., and E. A. TAYLOR, "The Salt-velocity Method of Water Measurement," *Jour. A.S.M.E.*, December, 1923.

1500 ft. long, as well as circular and rectangular penstocks of concrete. The salt tanks varied from 150- to 1200-gal. capacity, controlled by quick-acting valves. Pop valves were used to inject the solution into the penstock. Electrodes of various forms and sizes, of both steel and copper, were used, using one set in the circular penstocks for the downstream section and multiple sets covering the cross section in the rectangular penstocks. For recording the introduction of salt at the upstream section, both switches and electrodes were used. On the long penstocks a contact switch was installed on the handle of the quick-acting valve; on the short penstocks upstream electrodes were placed close to the pop valves. A recording ammeter with alternating current at 110 volts was used with, at times, portable transformers to change voltage and current in regulating needle deflection and height of curves. Time was recorded by either clock or field-made pendulum wired to the recording meters and by batteries, relay, and spark coil made to record jump-spark dots for each second on the chart.

In obtaining time of passage when a single switch on the quick-acting valve was used, time was computed for a point midway between the opening and closing of the valve. Where curves were made indicating the salt passing the electrodes, time was computed to the center of gravity of the curve or the center of the area. From 5 to 10 charges of salt solution were used for each run and their average results used in computing discharge.

These wheel tests were not checked by other methods except at one plant where the Gibson method was used and showed close agreement in results. Laboratory tests of this method made by Professor Allen, checking by both weir measurements and weighing tank, indicate a high degree of accuracy for this method when properly conducted, and it promises to be of great value in flume and penstock settings.

f. The pressure-variation method of Gibson¹ requires a penstock setting with length of pipe preferably not less than 100 ft. and a valve or turbine gate for quickly controlling flow. It requires the use of the Gibson apparatus for recording the changes of pressure at a point in the pipe line when the flow of water therein is gradually brought to rest by closing the turbine gates. This apparatus has a mercury U-tube to record photographically to full-scale fluctuations in pressure, also combining in the record the oscillations of a second pendulum, thus giving a complete pressure-change-time diagram. The mean change of pressure so recorded is a precise measure of the velocity destroyed, which with the corresponding value of Q may be determined by computation from the diagram. This method has the advantage that a relatively small change in velocity can be made to produce a relatively large change of pressure, and, hence, this element can be very accurately determined.

¹ GIBSON, N. R., "The Gibson Method and Apparatus for Measuring the Flow of Water in Closed Conduits," *Jour. A.S.M.E.*, December, 1923.

As the time element can be very closely measured, the resulting accuracy of the method is high. It is complicated, however, requiring special apparatus and can be used only in penstock settings. It has been used especially by Gibson in testing the large wheel units of the Niagara Falls Power Company.

g. Other Methods.—The color-velocity method, similar in principle to Allen's salt-velocity method but relying upon visual observation of the time of passage of the color solution, has been used in a few cases but requires a fairly long penstock for good results.

The venturi meter is accurate and, when installed, makes an excellent means of continuously checking up wheel efficiencies. It is too expensive to install, however, for acceptance tests alone. An adaptation of this method using as a venturi tube the contraction of the penstock at entrance to the wheel scroll casing and rating this tube as to discharge by the

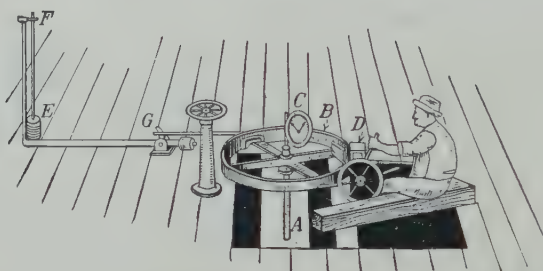


FIG. 89.—Holyoke testing flume—arrangement of dynamometer.

salt-velocity method, has been successfully used at the Davis Bridge (Harriman) plant¹ and thus affords an excellent means for obtaining operating records of use of water at this plant.

The moving-screen method, used somewhat abroad, is essentially a laboratory device, rarely used in this country.

Floats may occasionally be used in canals with low velocities or in special cases but are relatively undesirable, as a rule, as compared with the use of current meter.

B. 1 and 2. Dynamometer Measurements.—The arrangement of the Prony brake or dynamometer as used at the Holyoke testing flume is shown in Fig. 89, as used for testing a vertical-wheel setting. The wheel runner is set in an open wheel pit, and an extension of the wheel shaft *A* terminates in a brake pulley *B* with revolution counter *C*. The brake band around the pulley is tightened by hand wheel at *D*, so as just to balance the torque of the wheel by the weights on the scale arm at *E* by means of a pointer and scale at *F*. The bell crank at *G* gives a lever ratio usually about 10 to 1.

¹ *Jour. Boston Soc. Civil Eng.*, January, 1925, pp. 31–32.

For turbine tests in place the Alden dynamometer is best adapted.¹ This is in principle a Prony brake but is especially adapted for power measurement of units beyond the capacity of the ordinary Prony brake. Its general arrangement is shown in Fig. 90 with a sectional detail in

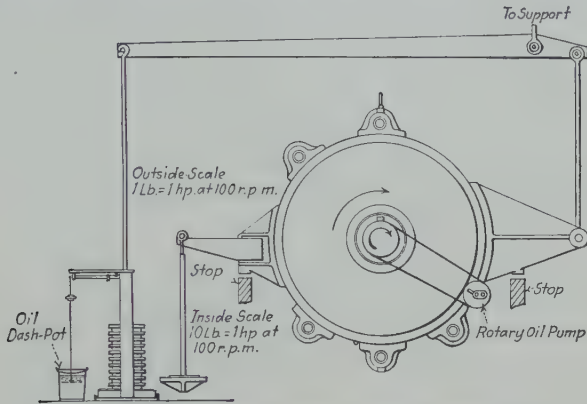


FIG. 90.—Alden dynamometer—general arrangement.

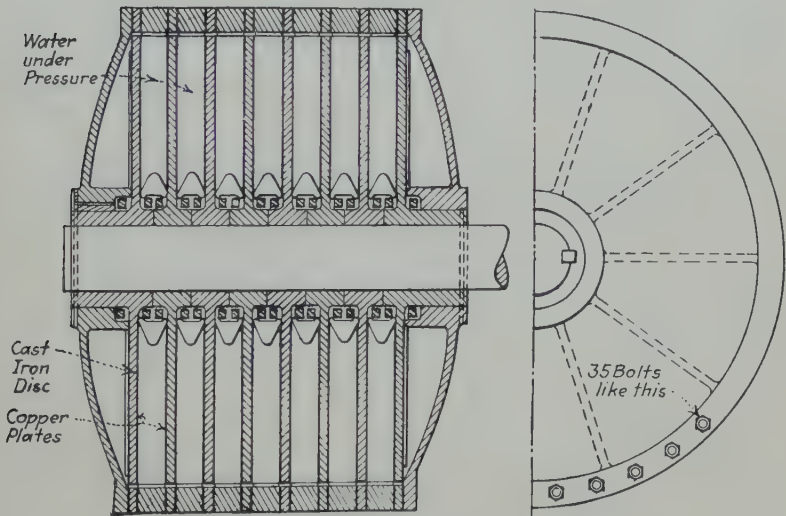


FIG. 91.—Alden dynamometer. Sectional detail.

Fig. 91, and it consists of several smooth circular revolvable cast-iron disks keyed to the shaft of the dynamometer, which is itself direct connected with the shaft of the wheel being tested. A non-revolvable housing bears upon the hubs of the revolving disks, and integral with this housing is a pair of thin copper plates in contact with each cast-iron disk.

¹ ALLEN, C. M., "Testing of Water Wheels after Installation," *Jour. A.S.M.E.*, April, 1910, p. 481.

Water is piped under pressure (or often piped from the forebay or flume level) so as to circulate through chambers between the units, each consisting of a disk and its copper plates and between the outer plate at either end and the wall of the housing. Another pipe system circulates oil, by a pump belted as shown, for lubricating the surface of the copper plates next to the revolving disks.

The water pressure tends to force the copper plates against the sides of the revolving cast-iron disks, thus creating friction and a load on the dynamometer. The oil on the disks and the passage of cool water permit the absorption of the energy used without excessive rise of temperature. The power transmitted from the wheel tends to rotate the housing, which tendency is weighed by the scale under the end of the long lever (Fig. 90). The weight pan with shorter lever arm, connected with the dynamometer at the left, is used to balance the dynamometer under no load (the initial weight) and to determine accurately small residual weights.

The Alden dynamometer has been extensively used by Professor Allen in testing hydraulic turbines and for wheel units varying in size from $\frac{1}{2}$ to 5000 hp. at speeds as high as 3600 r.p.m. (for small-sized brakes). Peripheral speeds of the cast-iron disks as high as 2 miles per minute have been used, but for safety usually the speed is limited to half that value.

In testing vertical wheels a knife-edged bell crank is used to transform the horizontal force into a vertical one in order to measure it on platform scales. Vertical units up to about 2500 hp. have been tested.

There are usually four disks in the larger machines, although from one to six disks have been used. The largest machine used has four disks, 60 in. in diameter, capable of handling 3000 to 3500 hp. at from 200 to 300 r.p.m. Using this with the next in size of 2500 hp. would enable a total of 6000 hp. to be held. Actually, units developing as much as 5000 hp. have been tested.

During the past 20 years between 300 and 400 field tests have been made under the direction of Professor Allen.

B. 3. Speed.—This is determined in r.p.m. by revolution counter or, if desired, as at Holyoke, with tachymeter showing total r.p.m. at any instant.

Value of Turbine Tests.—The advisability of testing turbines, either for acceptance or later to check up operating efficiency, will depend upon conditions, especially the value of the power generated. In the typical case of a modern plant in a power system, where power is valuable, it will nearly always pay to make such tests and furthermore, in the design and construction of new plants, provision should be made as far as practicable for making such tests. Thus, often at little or no increase in construction cost, the tailrace or canal can be provided with a suitable section for current-meter measurements or provision made, in the

case of penstock settings with fittings, for Pitot tube or salt-velocity measurements.

Holyoke tests are always of especial value in that they give complete wheel characteristics and are of unquestioned accuracy. A test in place under a head much greater than the 17- or 20-ft. head available at Holyoke will usually show a somewhat greater efficiency than the Holyoke test, due to the relatively greater mechanical losses at Holyoke. The Holyoke test is, however, a standard and always furnishes an excellent basis for specification requirements or comparison with other wheels there tested.

GENERAL ARRANGEMENT OF WATER POWER DEVELOPMENTS

Essential Features.—A water power development is essentially to utilize the available power in the fall of a river, through a portion of its course, by means of hydraulic turbines, which, as previously explained, are usually reaction wheels except for high heads, where impulse wheels may be used. To utilize its power, water must be confined in channels or pipes and brought to the wheels, so as to bring into action upon them substantially the full pressure due to the head or fall utilized, except for such losses of head as are unavoidable in bringing the water to the wheels.

The essential features of a water power development are therefore:

1. *The dam*, the structure of masonry or other materials built at a suitable location across the river, both to create head and to provide a large area or pond of water from which draft can readily be made. In many cases the power development is at or close to the dam, and the entire head utilized is that afforded at the dam itself, in which case the development is one of concentrated fall.

2. *The Waterway*.—More often the development must be by divided fall, utilizing in addition to the head created by the dam an additional amount obtained by carrying the water in a waterway, which may be a canal, penstock (or closed pipe), or a combination of these, for some distance downstream.

3. *The power house and equipment*, which include the hydraulic turbines and generators and their various accessories and the building required for their protection and convenient operation. Many existing water power developments also utilize the power from the wheels in mechanical drive, *i.e.*, operating machinery directly or by belting and gearing. The tendency is, however, very markedly toward mill and factory electrification, so that nearly all of the newer developments are hydroelectric.

4. *The tailrace* or waterway from the power house back to the river. In many cases the power house is located on the river bank so that no tailrace channel is required, but occasionally to develop additional fall a tailrace channel of some distance is used.

Gross and Net Head.—The development losses, as they may be called, aside from losses at or in the wheels, will vary in percentage amount depending upon the head and manner of development but should not usually exceed perhaps 5 to 10 per cent at the most.

The gross head developed is the fall between pond level at the dam and river level at junction with tailrace, or, in other words, the amount of fall of the river that is developed. The net or effective head (as previously explained on page 166) is less than the gross head by the losses sustained in bringing water to the wheels and, possibly, between tailrace level and river below. As previously noted, wheel efficiencies from 85 to 90 per cent or more are now obtained, so that a modern hydroelectric development should utilize in power supplied to the generators at least 80 per cent of the gross head of the development or not less than 75 per cent of the gross head at the switchboard.

Essentials of General Plant Layout.—The two basic principles to be kept in mind in planning a water power development are *economy* and *safety*, or in other words a maximum of power output at a minimum of cost, but at the same time a safe and proper construction that can meet the exigencies of operation imposed by structures which control as far as may be but of necessity interfere somewhat with natural forces, variable and often large in amount and uncertain in regimen. The hazards due to floods, ice, etc. must be provided for not only from the point of view of safety but also to minimize interruptions in plant operation as far as practicable.

Owing to freedom from the uncertain and irregular natural forces to which a water power development must of necessity be subjected, steam-electric plants were formerly considered as more dependable prime movers, but the interruptions in service at steam plants in this country—particularly in the northeastern section—during the times of fuel shortage of 1918 and thereafter, when for times water power alone was available for use, and later continued high fuel costs, have materially changed our perspective in this respect. The trend of modern water power developments toward simple and effective layout and also the greater use of stored water have resulted in a better appreciation of the value and dependability of water power, when properly utilized.

Factors Affecting Economy of Plant.—The factors or conditions affecting the relative economy of a water power development may be divided into the characteristics of (1) site and (2) use and market.

1. The site characteristics are those particularly affecting the construction and operating cost of the plant and, therefore, the conditions which are most likely to decide first of all whether a site is worthy of development and, if so, the best manner of making this development.

These include geologic conditions as affecting available foundations for structures, particularly the dam, whose type may be thus determined.

The absence of suitable rock foundations for the dam may even prevent the utilization of a power site.

Topographical conditions are also of great importance in determining the dimensions of the dam and thus largely affecting its cost and the relative proportion of the fall or head to be developed by the dam or by waterway, as well as the manner in which the waterway may be constructed, whether canal or penstock or a combination of these.

The slope of the river is of importance, as affecting necessary length, cost of waterway, and amount of pondage obtainable at the dam.

The relation of head to discharge also greatly affects the desirability of a power development. For a given amount of available power, the greater the head as compared with the discharge, the less costly will be the development, owing to the greater capacity required for all of the features except the dam, as discharge increases. In general, therefore the higher head developments are always less expensive per horse power of capacity than those of lower head.

Storage possibilities at sites upstream are of especial importance, where storage cost is reasonable—which will usually require the use of the stored water at several power plants in order to lessen its cost at each plant—in increasing the dependability of the water power development and the proportion of its output which will be primary or dependable power.

Operating costs may also be affected by especial conditions which may prevail on a given stream. Thus, a stream subject to frequent floods or high-water periods may have the power at a given site frequently curtailed by backwater in the tailrace, and on such a stream flashboards on the dam may require frequent renewal. The presence of ice, particularly anchor or frazil ice on streams having numerous falls or stretches of quick water, also introduces troublesome problems of operation and often adds to its cost.

2. The characteristics of use and market include the conditions particularly affecting the sale price and value of the developed power.

Thus proximity to market is a vital consideration. A water power site may be capable of development at low cost, *viz.*, with advantageous natural features, but situated so far from any possible market as to be unworthy of consideration for development. In this respect, the radius of possible transmission of power is constantly growing, and today lines of 200 to 300 miles are quite common. On the other hand, to transmit power such distances economically requires relatively large blocks of power, and in any event the cost of transmission must be included in power cost in competing with steam-electric plants at a distance. The transmission of power across state lines is also in some cases hampered or prohibited by state laws, and in the case of the utilization in this country

of the St. Lawrence River power, developed at sites in Canada, political difficulties are presented which are not yet solved.

The cost of other power at the available market is of importance as affecting the sale price of water power. This other power is commonly steam-electric, whose cost is largely affected by fuel cost. Hence, much variation in cost of power will be found in different parts of the country, depending upon the distance that coal (or oil fuel, in many cases) must be transported, freight charges here constituting the important element.

Load factor as affecting the manner of use of power is of great importance, as certain features of the water power development, particularly the power house and equipment, vary in cost nearly inversely as the load factor. It is of advantage, therefore, to keep the load factor at a hydro-electric development as high as possible.

The variation in the relative economy of water power in different parts of this country has already been discussed (see page 12), and the general conditions there noted are due to the resultant effect of the various natural and regional factors discussed above.

Types of Water Power Developments.—No two water power developments which are exactly alike will probably ever be built, and every power site has its especial problems of design and construction which must be met and solved. We may, however, distinguish certain general types of plant layout consistent with the general site characteristics of importance—head, available flow, topography of river and vicinity, etc.—all more or less interdependent, which affect the manner of development, as well as those characteristics of market and type of load, etc., which affect the size of plant and number of its units.

Such a general classification of water power developments is given in Table 61, which also shows to what extent head and flow may vary or affect the use of the different type as given.

TABLE 61.—TYPES OF WATER POWER DEVELOPMENTS

Type of development	Range of head, feet	Range of flow, second-feet
A. Concentrated fall:		
1. Open flume.....	Up to 20	Not limited
2. Concrete scroll flume.....	20– 60	Not limited
3. Steel scroll flume.....	60–200	Not limited
B. Divided fall:		
1. Canal and open flume.....	Up to 20	Above 600 to 800
2. Canal and concrete scroll flume...	20–60	Above 600 to 800
3. Canal and short penstock, steel scroll flume	60–up	Above 600 to 800
4. Penstock, steel scroll flume.....	60–up	Below 600 to 800

It must be kept in mind that the numerical limits given in Table 61 are somewhat variable and elastic for both head and flow. Thus, an open-flume installation may in some cases be used for a head of more than 20 ft., and penstock developments are sometimes made for capacities in excess of 800 sec.-ft. The figures given represent ordinary or common usage.

In the case of A-3, *viz.*, concentrated fall with steel flume, the ordinary upper limit of head is placed at 200 ft., although a dam of that height

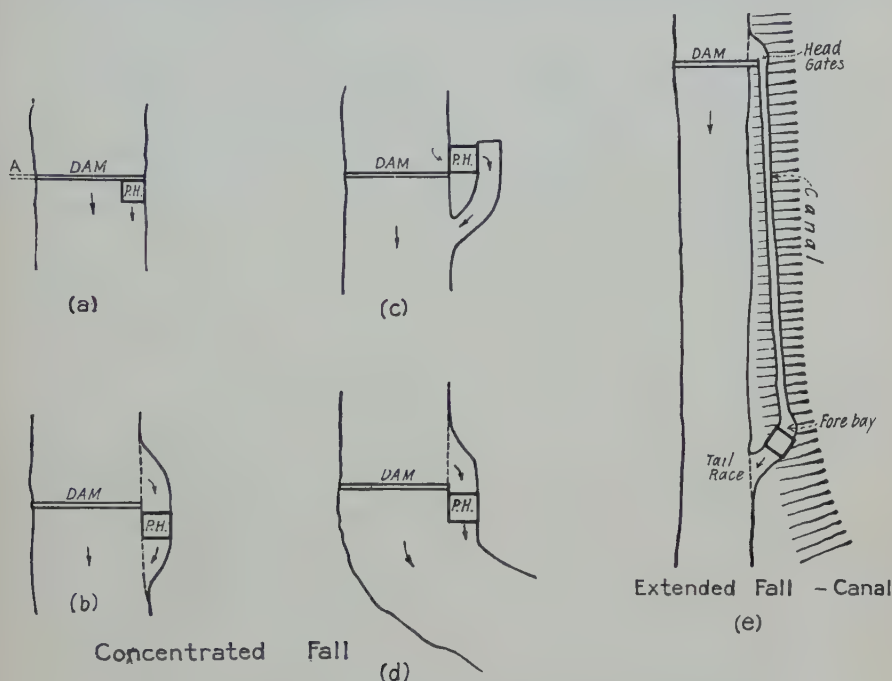


FIG. 92.—Arrangement of plants—concentrated and divided fall.

would seldom be economical for power development unless it afforded at the same time substantial storage capacity.

Typical Arrangements of Water Power Plants. A. *Concentrated Fall.*—The location of power house with reference to dam will depend upon local conditions. Often a low-cost development could be made by placing the power house in the river at one end of the dam (Fig. 92a). This would generally result, however, in an undesirable limitation in length of spillway and possible subjection of power house to flood and ice hazards. To obtain necessary spillway length, therefore, the power house must often be located in some such manner as shown in Fig. 92b, c, or d.

A few developments utilizing concentrated fall have been made using a hollow concrete dam of the Ambursen type with power house in the dam.

B. Divided Fall.—Various typical plant arrangements with divided fall are shown in Fig. 92*e* and Fig. 93*f* to *k*. Aside from capacity in second-feet, to be handled, the dominating feature is the topography of the region adjacent to the river. Thus, in Fig. 92*e* the river bank remains high and affords room for a canal development, which with open wheel pit could utilize a head of only about 20 ft. but with concrete scroll flume settings might make it possible to use a head of 60 ft.

The arrangement in Fig. 93*f* is typical of many developments where flow is relatively large, where the river bank permits the use of a canal

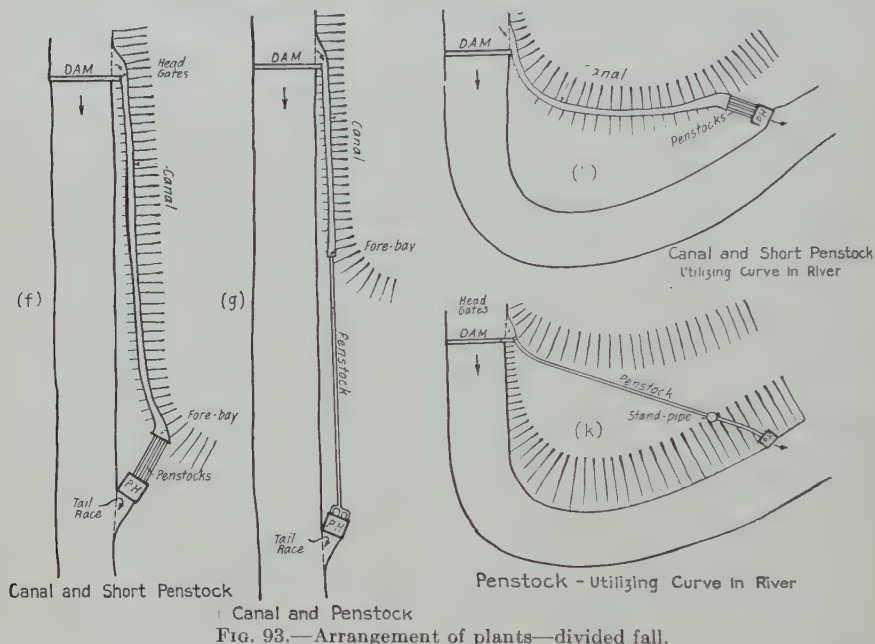


FIG. 93.—Arrangement of plants—divided fall.

to a forebay near the power house, from whence individual penstock lines run to each wheel unit. The head utilized in such a development will nominally be more than about 60 ft. and is limited above that amount only by the fall in the river between dam and tailrace level.

In Fig. 93*g* the topography is such that a canal can be used for only a part of the distance. If flow is large, it may be necessary here to use more than one penstock line, although such a development would result in increased cost, as compared with Fig. 93*f*, for a given total length of waterway.

In Fig. 93*h* the manner of development is similar to that of *g*, but advantage is taken of a bend in the river to utilize a greater head for a given length of waterway.

In Fig. 93*k* the flow is low enough to permit the use of a penstock throughout, which is kept at relatively high level to save in cost, until

near the power house, where a quick descent is made, usually with individual penstocks to each wheel unit. Here again a curve in the river is utilized to shorten the length of penstock.

A modification of Fig. 93*k* of service where the river bank between dam and power house site is very high, as with a hill, consists in constructing a tunnel penstock with surge tank and individual penstock lines to each unit from the point on the hillside where the tunnel emerges. The material most favoring tunnel construction is rock, and usually the tunnel would be lined to increase its flow capacity. The tunnel grade would be usually kept relatively flat, the sudden pitch being made with the penstock lines. The Davis Bridge development (see Chap. XIII) is of this type, although the tunnel in this case is under considerable pressure, due to location at a level about 100 ft. below that of the spillway, in order to utilize storage draft from the reservoir formed by the dam.

Lowest-cost Power Developments.—Keeping in mind variation in site, use, and market characteristics, it will be seen that the lowest cost development, as well as of power produced, will be secured with the following conditions:

CONDITIONS FAVORING LOW COST

(A Penstock Development)

1. Relatively high head and small flow.
2. Discharge assured by storage, the cost of which is carried by several plants.
3. Favorable dam site:
 - a.* Good foundations.
 - b.* Narrow valley and a minimum of material in dam.
4. Good penstock location, fairly straight line with moderate grade for most of distance, and then quick drop to power-house site.
5. A few large wheel units.
6. Relatively short transmission to market.
7. High load factor, often made possible where a plant is a unit of a large power system.

Highest Cost Power Developments.—Conversely, the highest cost development and of power produced will be for the following conditions:

CONDITIONS RESULTING IN HIGH COST

(A Canal Development)

1. Relatively low head and large flow.
2. Variable flow with small minimum or primary power.
3. Poor dam site:
 - a.* Poor foundations.
 - b.* Wide valley and relatively large material requirements.
4. Poor canal location, deep cut in hard material and circuitous route.
5. A relatively large number of small-capacity wheel units.
6. Long transmission to market.
7. Low load factor, as with an isolated plant and poor load characteristics.

Examples of Plants as Constructed.—In Chap. XIII are descriptions and plans of certain portions of seven typical water power developments for reference and study.

In studying these plants the student should first note the general method and type of development, considering and analyzing as far as practicable the characteristics of site and use which apparently led to the form of construction adopted.

The best method for general layout of a water power development is a matter worthy of much study and the most careful attention, particularly where a development covering a long stretch of river is planned, capable perhaps of several different schemes of procedure. It may be necessary to make tentative designs or studies and cost estimates for several different methods of development before proceeding with the final plans as adopted. Time should be taken for this to the full extent necessary for an intelligent conclusion, as the general method once adopted in construction cannot be changed and it is likely to be of fundamental importance as to results which can be secured with the plant.

ADDITIONAL TURBINE TEST DATA

TESTING FLUME OF THE HOLYOKE WATER POWER COMPANY, HOLYOKE, MASS.

Report of tests of a: 34-in. right-hand N.X. Allis-Chalmers turbine wheel.

Test No: 2905, made Mar. 21, 22, 23, 1923

Swing gate: Hydracone draft-tube Clearance: 15 in.

Number of experiment	Proportional parts of		Head acting on the wheel, feet	Revolutions of the wheel per minute	Relative velocity based on diameter of 34 in.	Quantity of water discharged by the wheel, cubic feet per second	Power developed by the wheel, horse power	Efficiency of the wheel, per cent
	Opening of the speed gate, inches	The full discharge of the wheel						
36	4.75	0.911	14.82	338.00	1.480	92.43	128.83	82.93
34	4.75	0.924	14.80	326.75	1.571	93.68	133.17	84.69
33	4.75	0.943	14.76	352.75	1.698	95.50	138.09	86.38
32	4.75	0.972	14.66	383.00	1.850	98.14	143.77	88.11
35	4.75	0.994	14.59	408.00	1.976	100.11	147.68	89.16
31	4.75	1.011	14.55	424.00	2.056	101.62	147.79	88.13
37	4.75	1.013	14.48	425.00	2.066	101.62	148.14	88.77
30	4.75	1.041	14.45	454.25	2.370	104.32	146.15	85.49
29	4.75	1.080	14.33	497.00	2.429	107.76	146.58	83.70
41	4.625	0.948	14.85	374.00	1.795	96.29	142.40	87.81
43	4.625	0.963	14.76	388.75	1.872	97.56	143.84	88.08
40	4.625	0.967	14.75	393.67	1.896	97.91	143.55	87.65
39	4.625	0.981	14.57	408.00	1.977	98.72	142.21	87.18
42	4.625	0.985	14.70	417.25	2.013	99.52	145.44	87.66
27	4.500	0.889	15.04	308.33	1.471	90.87	128.97	83.21
26	4.500	0.898	15.03	320.00	1.527	91.76	132.13	84.48
22	4.500	0.914	14.98	342.50	1.637	93.22	135.91	85.82
25	4.500	0.915	14.94	343.50	1.644	93.22	136.31	86.30
21	4.500	0.930	14.93	373.50	1.788	95.38	142.20	88.05
28	4.500	0.951	14.92	390.50	1.870	96.87	145.54	88.79
24	4.500	0.952	14.85	393.25	1.888	96.76	143.40	88.00
23	4.500	0.968	14.80	412.33	1.983	98.14	143.72	87.25
20	4.500	1.000	14.61	444.00	2.149	100.81	142.86	85.53
19	4.500	1.032	14.50	479.60	2.330	103.61	141.45	83.02
18	4.500	1.068	14.38	514.25	2.508	106.80	137.88	79.16
17	4.25	0.871	15.04	310.50	1.481	89.08	128.21	84.38
16	4.25	0.890	14.99	335.20	1.601	90.87	133.01	86.11
14	4.25	0.901	14.85	353.20	1.695	91.54	134.48	87.23
13	4.25	0.910	14.80	373.00	1.793	92.32	136.01	87.78
15	4.25	0.933	14.86	399.33	1.916	94.82	139.18	87.10
12	4.25	0.955	14.69	427.50	2.063	96.53	137.55	85.53
11	4.25	0.985	14.58	462.75	2.242	99.18	136.48	83.22
10	4.25	1.015	14.49	496.67	2.414	101.89	133.17	79.53
9	4.00	0.821	15.37	299.67	1.469	85.65	123.41	82.88
8	4.00	0.830	15.33	311.00	1.605	87.09	129.07	85.58
7	4.00	0.845	15.27	339.00	1.711	88.19	131.64	86.42
6	4.00	0.857	15.23	361.00	1.815	89.63	133.26	86.31
5	4.00	0.872	15.19	382.33	1.978	91.76	133.85	84.95
4	4.00	0.895	15.14	416.00	2.112	93.45	130.73	81.85
3	4.00	0.913	15.07	443.25	2.294	96.07	128.62	78.91
1	4.00	0.942	14.96	479.67	2.468	99.06	124.27	74.24
2	4.00	0.973	14.90	515.00				
Draft-tube Clearance 13 In.								
47	4.625	0.935	15.25	353.80	1.723	96.29	144.36	86.69
46	4.625	0.962	15.22	395.00	1.872	98.95	150.39	88.05
48	4.625	0.976	15.11	406.50	1.934	99.99	154.41	87.78
45	4.625	0.981	15.13	414.00	1.969	100.57	150.96	87.48
44	4.625	0.994	15.03	424.00	2.023	101.62	147.79	85.32
Draft-tube Clearance 17 In.								
73	5.25	0.855	14.77	338.00	1.627	98.02	134.13	81.69
72	5.25	0.872	14.70	351.60	1.745	99.76	137.67	82.78
71	5.25	0.885	14.64	383.83	1.856	101.04	139.96	83.43
69	5.25	0.922	14.52	414.50	2.012	104.91	144.48	83.63
70	5.25	0.951	14.39	440.75	2.149	107.84	147.72	84.09
68	5.25	1.000	14.41	486.25	2.369	113.30	156.45	84.50
67	5.25	1.041	14.20	525.00	2.577	117.09	154.84	82.12
74	5.25	1.064	14.00	545.25	2.696	118.81	146.19	77.50
63	5.00	0.829	14.86	323.20	1.551	95.38	133.45	83.02
62	5.00	0.846	14.81	347.50	1.670	97.22	137.90	84.45
60	5.00	0.862	14.77	370.50	1.783	98.83	141.06	85.21
59	5.00	0.888	14.69	392.33	1.893	101.62	143.06	84.50
61	5.00	0.883	14.67	394.50	1.905	100.92	143.85	85.68
64	5.00	0.894	14.64	405.40	1.960	102.09	144.57	85.29
66	5.00	0.924	14.60	432.00	2.091	105.38	150.28	86.30
58	5.00	0.927	14.57	437.50	2.120	105.62	152.50	87.38
65	5.00	0.943	14.43	450.00	2.191	106.92	152.03	86.88
57	5.00	0.965	14.40	473.50	2.308	109.31	152.35	85.34
56	5.00	0.999	14.17	505.00	2.482	112.21	148.94	82.60

TESTING FLUME OF THE HOLYOKE WATER POWER COMPANY, HOLYOKE, MASS.—
(Continued)

Number of experiment	Proportional part of		Head acting on the wheel, feet	Revol- utions of the wheel per minute	Relative velocity based on diameter of 34 in.	Quantity of water discharged by the wheel, cubic feet per second	Power developed by the wheel, horse power	Efficiency of the wheel, per cent
	Opening of the speed gate, inches	The full discharge of the wheel						
55	5.00	1.035	14.01	539.33	2.665	115.62	144.61	78.72
52	4.625	0.831	14.85	364.00	1.747	95.61	138.59	86.07
51	4.625	0.846	14.84	385.00	1.849	97.22	142.45	87.06
53	4.625	0.851	14.79	391.25	1.882	97.68	144.77	88.36
50	4.625	0.856	14.81	398.50	1.915	98.37	145.31	87.95
54	4.625	0.873	14.64	417.25	2.017	99.64	145.44	87.91
80	3.50	0.683	15.44	307.00	1.445	80.09	108.65	77.48
79	3.50	0.685	15.43	326.65	1.536	80.31	110.22	78.43
77	3.50	0.684	15.47	338.00	1.590	80.31	108.75	77.18
81	3.50	0.684	15.41	338.67	1.596	80.09	108.97	77.85
84	3.50	0.686	15.41	348.00	1.640	80.42	110.10	78.34
78	3.50	0.694	15.38	360.20	1.699	81.28	111.06	78.34
76	3.50	0.704	15.41	381.33	1.797	82.47	112.47	78.03
75	3.50	0.715	15.33	407.00	1.923	83.57	109.13	75.11
82	3.50	0.738	15.20	446.25	2.117	85.87	107.68	72.75
83	3.50	0.758	15.12	478.83	2.278	87.97	102.71	68.09
91	3.00	0.613	15.85	302.75	1.407	72.89	92.54	70.63
90	3.00	0.616	15.83	323.00	1.502	73.10	93.53	71.27
89	3.00	0.621	15.80	346.25	1.611	73.72	94.69	71.69
88	3.00	0.627	15.79	367.50	1.711	74.35	94.59	71.06
87	3.00	0.630	15.73	383.40	1.788	74.56	92.52	69.56
92	3.00	0.650	15.70	408.67	1.908	76.88	93.14	68.04
86	3.00	0.657	15.58	430.50	2.017	77.42	92.34	67.50
85	3.00	0.675	15.49	467.00	2.195	79.34	87.65	62.89
93	3.00	0.458	16.45	Still		55.48		
98	3.00	0.807	15.45	665.00	3.129	94.70		
96	4.75	1.286	13.80	765.00	3.809	142.64		
95	4.75	0.592	15.97	Still		70.62		
97	3.75	1.031	14.67	739.50	3.571	117.83		
94	3.75	0.519	16.24	Still		62.45		

NOTES: Turbine and dynamometer carried during test on ball bearing. With the flume empty, a strain of $2\frac{1}{2}$ lb., applied at a distance of 2.8 ft. from the center of the shaft, sufficed to start the wheel

HOLYOKE WATER POWER COMPANY.

A. W. LADD
Engineer in charge of experiments

A. F. SICKMAN
Hydraulic Engineer

TESTING FLUME OF THE HOLYOKE WATER POWER COMPANY, HOLYOKE, MASS

Report of tests of a: 30.2 in. right-hand Allis-Chalmers turbine wheel.

Using White's Hydraulone.

Test No.: 2959, made Apr. 14, 15, 1924.

Swing gate: Conical draft-tube.

Number of experiment	Proportional part of		Head acting on the wheel, feet	Revolutions of the wheel per minute	Relative velocity based on diameter of 30.2 in.	Quantity of water discharged by the wheel, cubic feet per second	Power developed by the wheel, horse power	Efficiency of the wheel, per cent
	Opening of the speed gate, inches	The full discharge of the wheel						
123	3.75	0.964	16.24	121.50	0.495	106.80	135.95	69.11
122	3.75	0.967	16.24	135.25	0.551	107.16	143.37	72.63
121	3.75	0.970	16.14	151.50	0.620	107.16	147.21	75.04
117	3.75	0.978	15.53	163.50	0.682	105.97	144.43	77.88
118	3.75	0.988	15.60	131.20	0.754	107.28	149.39	78.70
119	3.75	1.005	15.66	208.50	0.866	109.43	153.48	78.97
120	3.75	1.019	15.79	233.25	0.964	111.36	151.10	75.76
116	3.25	0.884	16.21	127.00	0.518	97.91	134.62	74.79
113	3.25	0.890	16.36	145.75	0.592	99.06	145.91	79.38
114	3.25	0.896	16.41	162.00	0.657	99.87	152.64	82.12
113	3.25	0.905	16.44	181.00	0.733	100.92	159.89	84.97
112	3.25	0.918	16.45	200.25	0.811	102.44	165.10	86.38
111	3.25	0.930	16.09	212.00	0.868	102.56	162.30	86.72
110	3.25	0.939	16.10	234.25	0.959	103.61	158.64	83.85
109	3.25	0.932	16.22	255.50	1.042	103.26	142.94	75.25
107	3.25	0.913	16.35	267.50	1.087	101.51	126.02	66.95
108	3.25	0.909	16.30	261.33	1.145	100.92	107.69	57.72
56	2.75	0.783	17.74	117.25	0.457	90.75	133.26	72.98
55	2.75	0.796	17.74	143.75	0.561	92.21	152.38	82.13
54	2.75	0.800	17.75	159.25	0.621	92.66	159.43	85.46
53	2.75	0.803	17.76	174.40	0.680	93.11	164.33	87.62
52	2.75	0.810	17.74	194.00	0.757	93.79	171.37	90.81
51	2.75	0.814	17.69	203.20	0.794	94.13	173.51	91.87
50	2.75	0.815	17.70	210.75	0.823	94.25	173.75	91.83
57	2.75	0.817	17.64	216.25	0.846	94.36	171.92	91.08
58	2.75	0.816	17.64	228.00	0.892	94.25	167.84	89.02
59	2.75	0.809	17.64	242.00	0.947	93.45	156.76	83.84
60	2.75	0.802	17.66	255.33	0.998	92.66	142.85	76.96
49	2.50	0.751	17.80	134.00	0.522	87.20	142.04	80.68
48	2.50	0.755	17.80	151.50	0.590	87.64	151.67	85.72
47	2.50	0.765	17.78	167.50	0.653	88.75	157.83	88.78
45	2.50	0.767	17.76	187.00	0.729	86.86	165.19	92.29
40	2.50	0.768	17.49	190.20	0.747	88.30	162.41	92.72
46	2.50	0.767	17.78	195.25	0.761	88.97	166.72	92.92
44	2.50	0.768	17.72	196.00	0.773	88.97	166.16	92.92
41	2.50	0.768	17.46	197.25	0.776	88.30	162.62	93.00
39	2.50	0.767	17.51	203.00	0.797	88.30	161.39	92.03
42	2.50	0.765	17.62	210.00	0.822	88.30	160.77	91.11
38	2.50	0.763	17.64	214.50	0.839	88.19	157.90	89.49
37	2.50	0.760	17.82	228.50	0.889	88.19	154.75	86.82
36	2.50	0.757	17.92	248.75	0.965	88.19	146.49	81.72
35	2.50	0.759	17.92	270.00	1.048	88.41	135.15	75.21
34	2.50	0.756	17.76	285.25	1.112	87.64	117.59	66.61
71	2.45	0.748	17.83	181.00	0.704	86.86	162.55	92.54
70	2.45	0.749	17.80	185.00	0.720	86.86	163.42	93.19
72	2.45	0.747	17.62	183.75	0.719	86.20	159.61	92.65
69	2.45	0.749	17.79	188.33	0.734	86.86	163.59	93.34
68	2.45	0.748	17.77	190.75	0.743	86.75	162.88	93.16
80	2.37	0.712	17.65	111.75	0.437	82.26	118.46	71.93
79	2.37	0.728	17.48	143.75	0.565	83.68	139.68	84.19
78	2.37	0.731	17.15	156.50	0.621	83.24	142.85	88.23
75	2.37	0.737	17.18	177.25	0.703	84.01	151.35	92.46
77	2.37	0.738	17.15	181.25	0.719	84.01	151.57	92.75
74	2.37	0.738	17.27	186.00	0.735	84.34	153.35	92.82
76	2.37	0.737	17.02	185.83	0.740	83.68	150.47	93.15
73	2.37	0.736	17.44	193.80	0.762	84.55	154.07	92.13
81	2.37	0.734	17.47	205.80	0.809	84.34	151.49	90.65
82	2.37	0.730	17.26	213.00	0.842	83.46	144.25	88.29
83	2.37	0.732	17.16	235.00	0.932	83.35	138.39	85.31
84	2.37	0.730	17.21	235.00	0.931	83.24	138.39	85.17
85	2.37	0.730	17.16	253.75	1.006	83.13	127.02	78.51
105	2.25	0.696	16.88	117.75	0.471	78.70	114.42	75.94
104	2.25	0.708	16.95	145.00	0.579	80.20	132.35	85.84
106	2.25	0.713	16.98	164.75	0.657	80.85	140.68	90.35
101	2.25	0.717	17.41	176.25	0.694	82.26	150.50	92.65
103	2.25	0.717	17.15	178.25	0.707	81.72	146.96	92.45
100	2.25	0.715	17.20	185.50	0.735	81.61	147.47	92.63
102	2.25	0.714	17.31	191.00	0.754	81.72	149.04	92.89

TESTING FLUME OF THE HOLYOKE WATER POWER COMPANY, HOLYOKE, MASS.—
 (Continued)

Number of experiment	Proportional part of		Head acting on the wheel, feet	Revolutions of the wheel per minute	Relative velocity based on diameter of 30.2 in.	Quantity of water discharged by the wheel, cubic feet per second	Power developed by the wheel, horse power	Efficiency of the wheel, per cent
	Opening of the speed gate, inches	The full discharge of the wheel						
98	2.25	0.710	16.98	198.75	0.792	80.42	140.45	90.68
97	2.25	0.705	17.02	218.50	0.870	79.98	135.11	87.51
96	2.25	0.708	17.03	242.00	0.964	80.31	128.26	82.68
95	2.25	0.702	17.11	260.75	1.036	79.87	115.17	74.30
94	2.25	0.703	17.12	263.75	1.127	79.98	100.26	64.56
65	2.30	0.712	17.81	182.00	0.709	82.69	155.41	93.04
67	2.30	0.712	17.83	185.75	0.723	82.69	155.88	93.22
63	2.30	0.711	17.78	187.50	0.731	82.47	154.59	92.95
65	2.30	0.710	17.76	189.80	0.740	82.26	153.69	92.75
62	2.30	0.709	17.81	192.50	0.749	82.26	153.04	92.10
64	2.30	0.708	17.76	195.25	0.761	82.04	152.35	92.19
61	2.30	0.707	17.87	198.25	0.771	82.15	151.77	91.15
29	1.85	0.587	16.96	122.67	0.489	66.54	101.14	79.01
28	1.85	0.600	17.14	153.00	0.607	68.37	117.13	88.13
27	1.85	0.597	17.52	174.75	0.686	68.67	123.49	90.50
33	1.85	0.595	17.79	177.75	0.692	68.98	125.61	90.25
30	1.85	0.596	16.93	173.00	0.601	67.45	117.16	90.46
31	1.85	0.593	16.97	177.75	0.709	67.14	117.24	90.72
26	1.85	0.591	17.89	194.00	0.754	68.77	125.67	90.06
26	1.85	0.589	18.16	213.75	0.824	69.08	125.88	88.47
24	1.85	0.586	18.15	227.00	0.875	68.67	120.31	85.11
23	1.85	0.585	18.16	241.75	0.932	68.57	113.89	80.64
22	1.85	0.585	18.21	266.75	1.027	68.67	102.11	71.99
21	1.85	0.583	18.20	291.00	1.121	68.37	85.68	60.71
93	1.60	0.497	17.80	124.75	0.486	57.68	91.83	78.86
92	1.60	0.504	17.81	147.25	0.573	58.55	99.72	84.32
91	1.60	0.506	17.85	165.50	0.644	58.74	102.34	86.05
90	1.60	0.501	17.85	179.75	0.699	58.16	100.56	85.40
89	1.60	0.496	17.97	198.75	0.770	57.87	99.49	84.35
88	1.60	0.496	17.94	219.75	0.852	57.78	97.06	82.55
86	1.60	0.494	17.90	247.00	0.959	57.49	87.27	74.77
87	1.60	0.493	17.96	265.50	1.029	57.49	78.18	66.75
20	1.35	0.436	18.33	113.40	0.435	51.36	80.14	75.05
19	1.35	0.437	18.19	130.00	0.501	51.27	84.21	79.61
18	1.35	0.441	18.17	157.50	0.607	51.73	88.11	82.65
17	1.35	0.439	18.19	174.50	0.672	41.45	87.35	82.29
16	1.35	0.435	18.19	195.50	0.753	50.99	86.35	82.08
15	1.35	0.432	18.21	218.50	0.841	50.71	83.64	79.86
14	1.35	0.430	18.22	248.50	0.957	50.44	73.17	70.20
8	1.00	0.340	18.57	132.00	0.503	40.27	66.16	77.90
7	1.00	0.339	18.58	149.80	0.571	40.19	66.07	78.12
9	1.00	0.338	18.53	166.20	0.634	40.02	63.62	75.64
6	1.00	0.338	18.67	167.00	0.635	40.19	63.92	75.11
5	1.00	0.337	18.68	195.00	0.741	40.02	63.16	74.49
4	1.00	0.328	18.67	224.00	0.852	39.00	52.77	63.89
3	1.00	0.326	18.64	247.25	0.941	38.92	43.68	53.09
12	0.90	0.296	18.50	164.50	0.628	35.04	53.28	72.47
11	0.90	0.297	19.54	192.25	0.734	36.21	50.95	68.81
13	0.90	0.392	18.48	223.00	0.862	34.55	49.90	68.91
10	0.90	0.293	18.59	232.00	0.884	34.72	40.99	55.99
2	0.75	0.238	18.78	168.25	0.638	28.38	39.63	65.56
1	0.75	0.240	18.75	197.75	0.750	28.53	34.94	57.58
For remaining experiments, dynamometer removed.								
124	3.75	0.932	17.41	354.67	1.397	106.92		
125	3.25	0.857	17.48	361.00	1.419	98.49		
126	2.75	0.764	17.65	364.00	1.424	88.30		
127	2.50	0.719	17.74	365.67	1.426	83.24		
128	2.25	0.673	17.82	367.00	1.428	78.16		
129	1.75	0.529	18.11	355.00	1.371	61.96		
130	1.25	0.391	18.38	330.33	1.266	46.09		
131	0.75	0.277	18.58	293.67	1.119	32.80		

NOTES: Turbine and dynamometer carried during test on ball bearing. With the flume empty, a strain of 3 lb., applied at a distance of 3.0 ft. from the center of the shaft, sufficed to start the wheel.

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 Engineer in charge of experiments

HOLYOKE WATER POWER COMPANY.
 A. F. SICKMAN
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CHAPTER V

THE DAM

FACTORS AFFECTING LOCATION AND COST OF DAMS

The factors or elements affecting the location of a dam and its cost may be divided, for convenience, into items of (1) construction costs and (2) other costs.

1. Construction Costs.—Construction costs would include the following items, which are given in their usual order of importance. It should be remembered, however, that their importance will vary materially with the size of the dam and of the river on which it is located.

a. Foundation.—This is likely to be the most important item, affecting as it does the type of dam which may be used. Thus, without a ledge-rock foundation, a masonry dam of any considerable height is undesirable. The essentials for the foundation of any dam are that it is impervious or practically so and that it be able to withstand safely the weight of the dam, as well as any tendency to slide or shear.

A foundation of porous sand or gravel, with ledge either absent or at great depth, may present difficulties hard to overcome or surmountable only at great cost. On the other hand, ledge rock on one bank of the river may afford an opportunity for the spillway of the dam and the remainder of the structure may often be of earth, especially if the soil overlying the deeper rock is fairly impervious, so that a central core or section of fine material in the middle of the dam may be notched or trenched into it, so as to prevent seepage under and through the dam.

b. Topography as affecting the dimensions of the dam and quantities of materials of which it is composed, keeping in mind that material in the cross section of a masonry dam varies approximately as the square of its height. In general, a narrow valley is advantageous as to economy in quantity of materials, the extreme case of this being a narrow valley or canyon with rock sides where an arched dam may be used, requiring much less material than a gravity dam. The necessity of room for the spillway of the dam must usually be kept in mind, however, and this space may be difficult to obtain in a very narrow valley.

c. Materials for Construction.—A masonry dam, usually of concrete, requires suitable sand and stone or gravel within reasonable transportation distance of the site. Limitations of quality or distance of such materials or of distance and difficulty in transporting cement to a rela-

tively inaccessible job may prevent or limit the use of this type of dam and result in greater economy of an earth dam. Here again, suitable materials must be at hand, and it is obvious that the proper type of dam for a given site is often decided largely by the materials available for construction.

d. Handling Water.—Unwatering a stream and providing for carrying its flow past the site of a dam under construction are items whose importance varies greatly with the size of the stream. The flow of a stream with small drainage area may be handled with low cost—perhaps a few hundred dollars—often through a small temporary flume or even through an outlet pipe of the dam. A large stream may require expensive cofferdams which are subject to destruction by freshets and require a large outlay to provide adequately for contingencies during construction. As an illustration of this, the coffer for the Keokuk dam and work incidental to handling water constituted a very large item of cost. The methods employed at the Garvins Falls dam¹ were typical for a good-sized river.

A novel method² of constructing a closure cofferdam for a diversion channel was successfully used in 1930 upon the Chute à Caron development of the Aluminum Company of America, on Saguenay River, Quebec, consisting of building a precast concrete dam standing on end at the side of the main river channel and tipping it over into this channel at the proper time by blasting out a pier, so the precast dam would fall into an accurately predetermined position.

e. Accessibility of the site, both as to hauling in and storing of materials and, especially on larger projects, space for locating camp sites and construction equipment. In a narrow valley this item may cause some difficulty, and it is well for the engineer in planning a project to keep in mind probable necessities in this respect.

f. Interest during Construction.—This item covers interest charges between the time that construction expense is incurred and the plant is in operation. This will vary practically with the size of the project and the difficulty of construction, as affecting the time required to complete it, and, hence, may include interest charges for a season or longer.

g. Engineering and Contingencies.—Engineering costs include that of engineering supervision, preparation of plans, etc., which may vary from a small percentage in the case of large projects to 10 per cent or more in small projects requiring much detail in plans. A common allowance for this whole item—including probable contingencies not covered otherwise in estimates—is 15 per cent, which may be decreased or increased, depending on circumstances.

¹ SHEDD, G. G., *Jour. Assoc. Eng. Soc.*, October, 1905.

² "Blasting a Precast Dam into Place," *Civil Engineering, A.S.C.E.*, December, 1930, pp. 159-164.

2. Other Costs.—Other costs (*i.e.*, aside from plant-construction items):

a. Land and Real Estate.—This must include not only land and real estate which will be occupied or affected by the dam site itself and used for construction purposes but also that lying upstream along or near the river which may be affected by flowage due to the dam. Where a large reservoir is created by the dam, this may be an item of considerable importance.

Where a project involves power development, and land lying upstream from the dam site cannot be obtained by negotiation and purchase, it is possible under the Mill Act or its equivalent in certain states to take the land and let the courts fix its value. The dam site itself, however, must usually be purchased and owned by the parties causing flowage, prior to such takings.

In the case of a project involving storage alone, without power development connected thereto, the Mill Act or its equivalent does not apply, and this is one of the serious difficulties in the way of storage developments at present.

Another important phase of this subject is the possibility of securing legislation under which a storage reservoir company can compel the downstream owners to contribute to the cost of constructing and maintaining the reservoirs. This is extremely important, for there are many potential developments which are too expensive to be undertaken unless all those who receive the benefit contribute to the cost.

The problem of compelling mill owners to contribute to the cost of storage developed by others is, however, an exceedingly difficult one. It was solved in Wisconsin as to the Wisconsin River only by the unanimous consent of all parties in interest. New York has attempted to solve it by creating storage reservoir districts as municipal corporations and collecting the contributions from downstream owners in the form of taxes. Under this legislation in New York, such districts have been actually formed on several rivers and considered on others. Work has been done on Black River and the large Sacandaga project on Hudson River completed.

Other states are working on the same general problem and no doubt an adequate solution will be found, as it is of great importance in the development of water power.

b. Water Rights.—If developed water rights are affected by a flowage project, such rights must be purchased and are in the same category as the dam site. A possible exception to this may be noted in at least one state (New York), where a developed water power may be taken by a public utility corporation if required by "public necessity." This kind of action, however, has not as yet stood the test of legality in the higher courts.

Undeveloped power or water rights in the eastern states may be flowed under the Mill Act but with proper compensation to riparian owners.

In the western states, however, the principle has gradually been evolved that the waters of streams belong to the state and may be granted to an individual by the state.¹ Filings are made by posting a notice on the stream bank or by application to state engineer or water commissioners. The permit issued by the state is revocable and expires if use is not made of the water within a certain time. Interstate streams require agreements between the states, and care must also be taken to get any other necessary permits, as of the Federal Power Commission, or to cover locations in federal or state reservations.

The value of water rights will be further discussed later. This will vary with (1) the natural advantages of the site, as affecting the cost of development and (2) the proximity of the site to a good power market.

c. Railroad or Highway Relocation.—In a large project this may be an item of importance where railroads or highways would be affected by flowage from the dam. In fact, as railroads often follow rivers in their location, they frequently constitute serious obstacles to storage projects and often limit the manner of development of power projects, especially the extent of flowage from the dam.

Highways are more elastic as to grades and location than railroads but must often be relocated or raised in grade, particularly where the pond or reservoir formed by the dam is large in extent.

d. Miscellaneous Items.—These may include the cost of clearing the reservoir, removing any buildings, etc. In storage or pondage for power use it is usually only necessary to clear the reservoir site of trees and brush, and occasionally some salvage value in the form of lumber may be thus obtained.

A troublesome item sometimes met with, especially in storage projects, is the location of a cemetery within the desired limits of flowage. This may require regrading or even removal to another site and, owing to the complications of lot ownerships, often dating back many years, may result in serious difficulty and in some cases limit possible flowage.

Cost of Sacandaga Reservoir.—As an illustration of cost items for a large dam and reservoir project, Table 62 and data have been furnished through the courtesy of the Board of Hudson River Regulating District.² Further operating and maintenance cost data are given in Chap. XII.

The total actual cost of this project, completed in 1931, was a little over \$12,000,000, the increase in cost being due to extensive litigation with the Fonda, Johnstown and Gloversville Railroad, resulting in a delay

¹ See CHANDLER, A. E., "Elements of Western Water Law," 1913; and *Eng. News-Record*, Feb. 3, 1927, p. 204.

² See also "Water Power and Storage Possibilities of the Hudson River," p. 23, New York Water Power Commission, 1922.

TABLE 62.—SACANDAGA RESERVOIR

Total storage capacity of reservoir = 30 bill. cu. ft.; earth dam 90 ft. high, 1200 ft. long; spillway channel through ledge on left bank.

Estimate of Cost

Property damage including lands, telephone, telegraph, and transmission lines.....	\$2,350,000
Clearing of reservoir basin.....	710,000
Regulating dam and appurtenances.....	2,200,000
Relocation of Fonda, Johnstown and Gloversville R. R.....	400,000
New state highways 6.4 miles at \$50,000.....	320,000
New town highways 35 miles at \$18,000.....	630,000
Removing cemeteries.....	100,000
New bridges.....	360,000
Ferry at Northampton.....	50,000
Total.....	\$7,120,000
Engineering, general overhead, and contingencies of construction, 15 per cent.....	1,068,000
Total.....	\$8,188,000
Preliminary expenses.....	212,000
Interest during construction.....	350,000
Contingent fund.....	250,000
Total.....	\$9,000,000

of 3 years and consequent great increase in overhead costs and interest during construction. All construction work was done within the original estimates of cost.

The actual cost per million cubic feet of capacity was \$400.

INVESTIGATION OF DAM AND RESERVOIR SITES

This may, in general, be divided into preliminary and final investigations, although there is often considerable overlapping of work (or what may be called intermediate steps or studies) as between such divisions as assumed.

Preliminary Investigations.—These in general constitute such investigations as are required to establish the feasibility of the project or at least to ascertain that it merits further or more detailed investigations. In the preliminary stage several possible dam sites may be considered and such information gathered as will be helpful in limiting further investigations to one or more of these sites.

This will usually require a careful examination of each site to determine as far as possible, by observation of surface conditions, the character of the foundation and the materials available for construction. Approximate surveys will be required to give perhaps one or more cross sections

at the dam site, so that approximate estimates of quantities of earth and rock excavation, masonry, etc., may be made.

It is seldom possible to foretell from surface appearances exactly what will be met with in the foundation. On the other hand, estimates may often be made within limits, especially when an earth dam is planned and the character of soil fairly evident; or if ledge rock is exposed at numerous places on both sides of the river, so that there is little question as to its presence, within reasonable depth from the ground surface.

Other factors affecting cost as previously outlined should be kept in mind, and enough information obtained to cover them, at least within rough limits.

Usually, in the preliminary stage the necessary land for dam sites will be held under option, such options having been acquired before making any surveys and arranged to run for a sufficient length of time to permit of complete investigations and a decision as to what land will be needed in the project as finally developed.

Occasionally, it will be desirable in preliminary investigations to make test borings or dig test pits to determine foundation conditions. More often these will be a part of the final investigations.

Final Investigations.—The essentials in final investigations of a dam site will include:

1. The determination of which site is preferable where more than one is being considered.
2. Obtaining all necessary information for making plans and cost estimates of the dam, including foundation requirements.
3. Surveys, plans, and cost estimates of the reservoir or pond created by the dam or work required by it, such as railroad and highway relocation, etc.

1. *Alternate Sites.*—The decision between one or more sites is likely to be based upon relative foundation conditions as determined from test pits and borings, which are often not available from the preliminary work. These may consist of test pits or holes where the ledge overburden is not very deep or wash-drill borings or core borings extending into the rock, or combinations of these. Methods of investigating foundations will be discussed more in detail later.

Occasionally, in deciding between sites, it may be necessary to make more accurate surveys and plans than are available from the preliminary studies, in order to make better cost estimates.

2. *Data for Plans and Cost Estimates of Dam.*—Surveys of the dam site to obtain a topographic plan may be made conveniently with plane table or transit stadia with suitable control. The scale of the plan will depend somewhat on the type of dam and its size—usually from 40 to 100 ft. to the inch, with contour interval of 2 to 5 ft. Especial care should be taken to locate and show on the plan all ledge outcrop, the

shore line of the river and its elevation, the location of all borings and soundings, etc. If the river is large, any reefs or islands which may be useful in building cofferdams and unwatering the stream should be located.

Any buildings, roads, railroads, or other structures in the vicinity of the dam site should also be shown on the plan, both to indicate what will be affected by construction and to help in the ready use of the plan in the field. Frequently buildings in the vicinity of the dam site may be of service for quarters during construction, after which they may be removed or torn down.

The topographic plan of the dam site, together with the data from borings and soundings and notes regarding any special features, should furnish complete information for use in making contract plans and in estimating quantities of materials and costs.

3. *Surveys, Plans, Etc., of Reservoir Site or Flowage Area.*—The survey and resulting plan of the reservoir site or area to be flooded by a proposed dam should show:

a. The location of all structures affected by it, such as buildings, railroads, roads, and any other artificial structures to a level well above that of the contemplated flowage.

b. Topography, especially in the vicinity of the probable flow line of the reservoir or pond, to serve in locating this line on the ground.

In the case of projects with reservoirs or large ponds, it is usually desirable to plot area and capacity curves for use in determining the best height of dam. Figure 94 is an example of such curves for the reservoir on Sacandaga River, New York, for which cost data are given on pages 270 and 271.

The area curve is based upon points, each one of which is the area (planimetered from the topographic plan) within some contour. For illustration, with 5-ft. contour intervals these points would be available for each 5-ft. difference in elevation. A smooth curve drawn through these points makes it possible to determine areas with reasonable accuracy for, say, each foot difference in elevation and to compute capacity-curve points for these same elevations.

The topographic plan for a reservoir site can usually be effectively made by plane table with transit-stadia control. The scale used will vary from about 1 in. = 1000 ft. or more, for large reservoirs, to 1 in. = 100 ft. or less, for relatively small ponds. In making the topographic surveys the portion of the reservoir site in the vicinity of the proposed water level of full reservoir is of most importance. The mid-depth and lower portions of the site may be only approximately surveyed as a basis for the area and capacity curves. Oftentimes, too, it will not be planned to draw a reservoir more than to a certain depth, especially where power is to be developed at the reservoir site, and this will limit the need of accurate topography.

c. A land plan showing all parcels of land that will be required is also an essential. This may often be the same plan as that showing topography, on which will be shown all walls, fences, and land subdivisions, with names of owners.

When a project proceeds to the stage of land takings and construction, it will often be necessary to survey each plot of land taken, as a basis for record plans and deeds of taking.

Borings and Foundation Investigations.—It seldom occurs that a dam site shows ledge rock or outcrop at the surface for the entire distance across a river valley. Even in the favorable situation where outcrop is

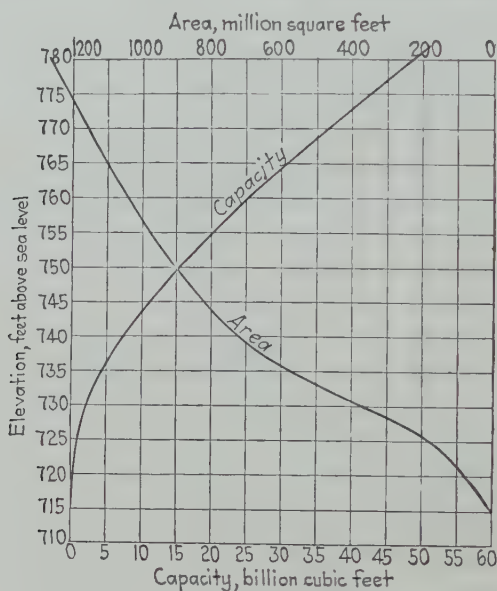


FIG. 94.—Sacandaga reservoir—area and capacity curves.

found on both shores, its elevation and condition in the river bed may not be apparent and may require borings to determine this without question.

More often, ledge rock, where existing, is at some depth below ground surface on one or both shores of the river and must be located and its condition ascertained before suitable estimates of quantities and cost can be prepared.

Where foundation other than rock is to be used, as, for illustration, hardpan or impervious material in the case of an earth dam, this foundation must also be explored to locate the impervious stratum and to make sure that it is continuous and not overlying pervious material.

Depending upon the conditions at the dam site under consideration, there are various methods of use in investigating foundation conditions.

Rod Soundings.—At slight cost, information of value may often be obtained by rod soundings, made by driving down a steel rod or bar by

means of a sledge. This is best done in sandy or soft material and is of chief value in determining the approximate thickness of such material. Thus, a site for an earth dam may have a layer of silt or soft material which must be removed, and rod soundings over the area involved (with perhaps a few check-test pits) will usually afford a good basis for estimating the amount of this soft material.

*Borings with Auger.*¹—Where ground material is free from boulders and not too hard, borings may be made with some form of earth auger, often to a depth of 15 to 30 ft., enabling samples of material to be obtained at different depths. Such an auger may be made by welding a carpenter's auger to a rod or pipe and works more efficiently if the centering point is cut off and the lips are shaped as shown in Fig. 95 (3).

A wrench formed of a plumber's tee and two short pieces of pipe may be used to screw the auger into the earth and pull it out with its load of material, and the rod may be lengthened by adding other sections of pipe as sinking progresses [Fig. 95 (1)].

Another much used type of auger is formed by making a spiral coil of tire iron, shaping a cutting bit on its lower end and welding or riveting the auger thus formed to a joint of rod or pipe [Fig. 95 (2)]. As the auger is heavy when loaded with material, a windlass or small derrick with pulley blocks may be used in lifting it and a platform may also be built from which the auger can be steadied and turned. In boring through dry sand or other loose deposits, a little water should be poured into the hole to cause the material to cling to the auger.

The ordinary post-hole auger is also useful in making borings in fine or soft material (or starting wash borings) and gives a continuous vertical sample of material. Under favorable conditions—in sand—depths of 25 ft. have been reached with this device.

Test pits—usually 5 or 6 ft. square, depending upon their depth—furnish an excellent means of determining ground materials, as the various

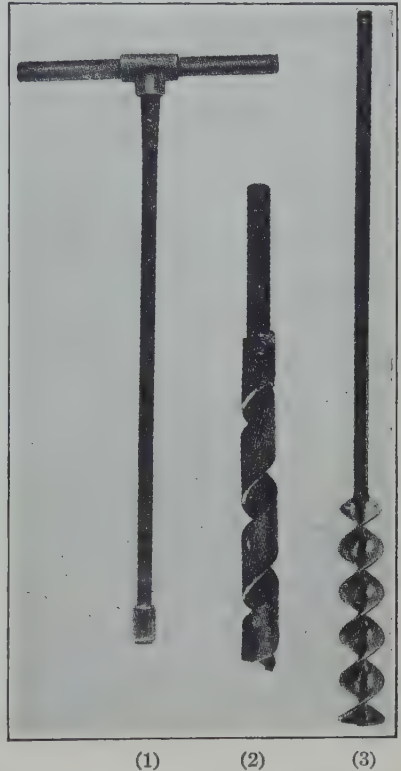


FIG. 95.—Earth augers.

¹ U. S. Geol. Survey, *Water Supply Paper* 257, pp. 88–90.

strata may be observed in practically an undisturbed condition. Without timber sheeting, a test pit cannot usually be more than 8 to 10 ft. deep

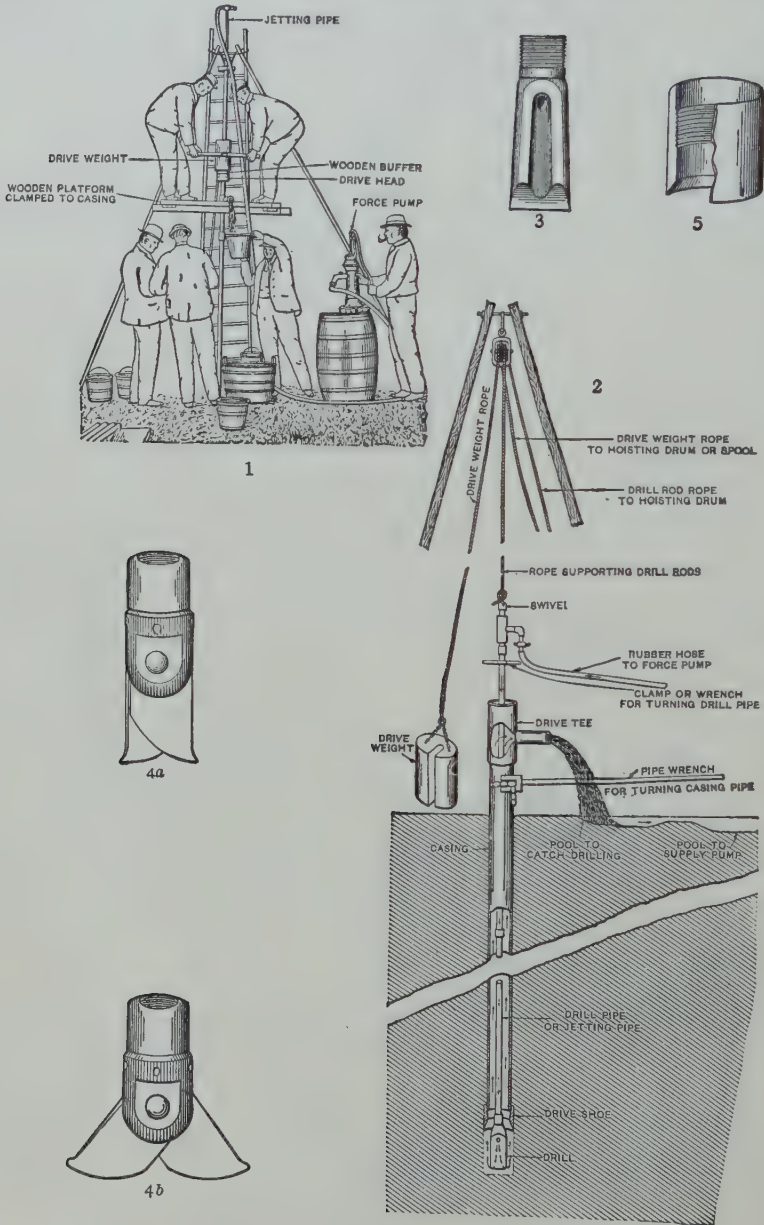


FIG. 96.—Wash drill outfit and method of use.

and the cost rapidly increases for greater depths. One or two deep test pits, however, even at considerable cost, may be well worth while.

Wash Borings (also called jetting or jet drilling).¹—The essential features of the wash-boring outfit are shown in Fig. 96. A drive pipe, usually $2\frac{1}{2}$ to 3 in. diameter, capped with a driving tee or casing, is sunk by weight or heavy hammer, either hand or machine operated. Inside the drive pipe is the drill pipe (or wash pipe), usually $\frac{3}{4}$ to 1 in. in diameter, with rubber-hose connection at the top to water pressure, usually from a force pump. The bottom of the drill pipe terminates in a drill point [Fig. 96 (3)] chisel shaped and with small hole connecting with the inside of the pipe, so that water will flow down through the drill pipe, through the hole in the drill, and up between outside of drill pipe and inside of pipe casing, carrying along the material stirred up by the drill, which flows out with the water through a side outlet or tee in the casing.

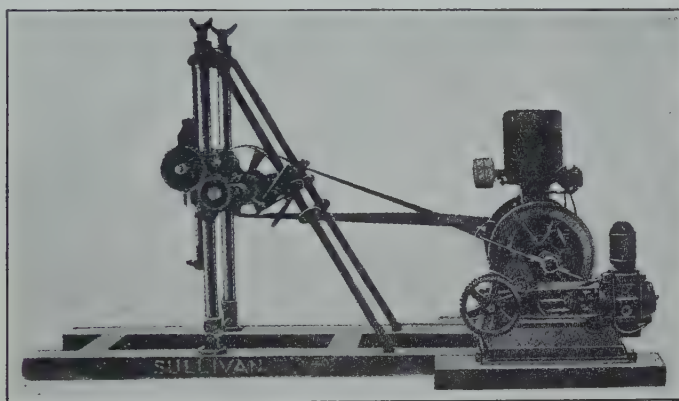


FIG. 97.—Portable diamond drill outfit. (Courtesy of Sullivan Machine Company.)

Samples of material are caught in a pail from time to time and kept for reference, noting the depth reached at the time.

The pipe casing is turned from time to time, and the process is fairly continuous, except as additional lengths of casing and drive pipe (each usually 5 or 6 ft. long) are required.

The samples from wash borings, of course, only furnish general information as to the kind of material, as the flowing water breaks up the existing formation and some fine material is inevitably washed away and lost. A skilled operator, however, from the samples obtained can make a fairly good interpretation of the results of such a boring, although an occasional test pit, getting actual samples of material to supplement the wash borings, is advisable.

Where boulders are found, it becomes difficult and often impossible to make wash borings, although progress can sometimes be made by pulling the casing a few feet, putting down a charge of dynamite, and

¹ U. S. Geol. Survey, *Water Supply Paper* 257, pp. 75–78.

blowing out or shattering the boulder so that the casing can be forced past it.

Wash borings are useful in that they can be quickly made but frequently must be accompanied by core borings to be of much value.

*Core Borings.*¹—Core borings are commonly either diamond drill or shot drill and, while costing usually four or five times as much as wash borings, have the advantage that an accurate sample is obtained of the material penetrated.

Diamond-drill Borings.—This outfit usually requires a derrick, hoisting sheave and drum, drill rod with hose connection similar to that

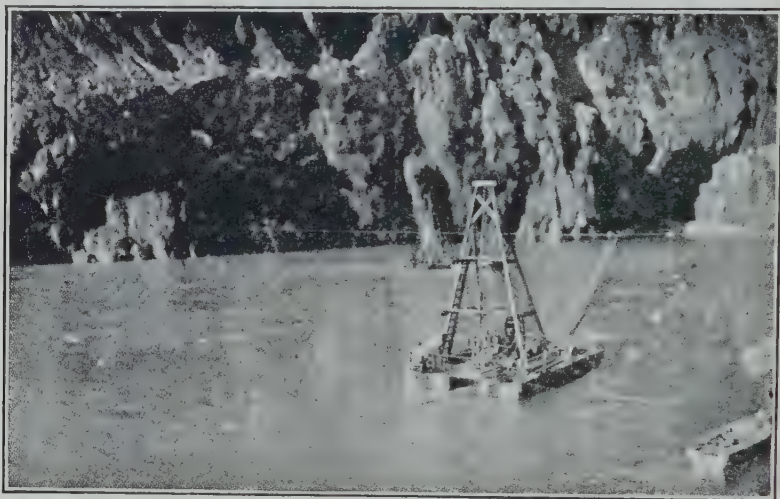


Fig. 98.—Diamond drill outfit in use on Colorado River. (Courtesy of Sullivan Machine Company.)

used for wash borings, rotating device (usually with bevel gears) for drill rod, feed mechanism, and cutting bit (see Figs. 97 and 98). Commonly a belt drive is used to rotate the gearing so that the belt may slip if the drill catches, thus avoiding twisting or breaking the rods. The drill rod must be fed forward by either hydraulic piston or screw feed, so that the rods will always be supported, even in soft rock, and the total weight of rods not come upon the diamonds.

The bit [Fig. 99 (3)] is made of a ring or sleeve of tough but ductile steel, $\frac{3}{8}$ to $\frac{5}{8}$ in. thick, whose upper end is threaded to screw into the drill rods and whose lower end is turned true and bored with eight holes to contain the diamonds. The diamonds are carefully set in the holes so that their lowest points lie in a plane and are arranged alternately on the inner and outer faces of the bit, so that the edges of the outer four project a little beyond the outer face and give it clearance, while the edges

¹ U. S. Geol. Survey, *Water Supply Paper* 257, pp. 78–88.

of the inner four project slightly beyond the inner face and clear the drill of the core. The diamonds are calked in place by carefully swaging the steel firmly against them with light blows, or the diamonds are set in steel nibs which are soldered or brazed into their places in the bits.

The shape of the bit and arrangement of the diamonds permit the stones to cut a path for the drill and slightly enlarge the cavity around it, and this clearance allows the drill to be easily lowered into the hole and also gives passage between the rods and wall of the hole for the drill water to convey the drillings to the surface. The stream of water also keeps the bit cool and prevents injury to the diamonds by heating.

In starting a hole with a diamond drill, the first requirement is to get down to rock. If the soil is thin, a pit is dug through it, a drive pipe is inserted, and a tight-fitting joint is made by chiseling out a seat in the rock, driving the pipe down, and calking it firmly. If the soil covering is more than 10 or 12 ft. deep, it is cheaper to drive the pipe to rock than to dig a pit for it. In places where the drive pipe cannot be set into the rock with sufficient firmness to keep out the surface water, a hole is drilled inside the drive pipe with a chopping bit [Fig. 99 (7)] attached to the drill rods, and a string of casing is put down to a depth sufficient to exclude the water.

When ready to begin drilling, the drill spindle is set to the angle desired for the hole (usually vertical), a bit and core barrel are screwed to the lower end of a joint of drill rods, and other rods are added so as to bring the bit to the rock. The upper length of rod is then clamped in the spindle, so that the bit is a little above the rock, and the water swivel is screwed to the top of the rods and connected with the pump. The pump is then started, and, when the return water begins to flow from the top of the casing, the drill is started and the bit is slowly run down to rock. If the rock surface is not smooth, the bit will at first drill only one side, but soon all of the diamonds are cutting the rock, or, as the driller puts it, the drill has an "even bearing."

When the drill has been run into the rock the length of the core barrel, drilling is stopped, but the pump is kept running long enough to wash all cuttings to the surface. A hoisting plug is then substituted for the water swivel, and the line of rods is hoisted until the first drill-rod coupling is just above the safety clamp [see Fig. 99 (6)]. This clamp allows the rods to be hoisted, but grips them and guards against their loss by slipping downward and dropping into the hole. The top joint of the rod is then unscrewed and passed out through the hollow spindle, the hoisting plug is lowered and screwed into the second length of the drill rods, and the line of rods is hoisted another length. Each joint in turn is removed until the core barrel reaches the surface, and the core is then taken out. By reversing the process the rods are again lowered into the hole, the water swivel is connected, and drilling is begun on the next "run."

Shot-drill Borings.—The shot-drill outfit is similar to the diamond drill, but the cutting is accomplished by a short length of steel tube with one or more notches or slots in the cutting edge. Under the weight of the

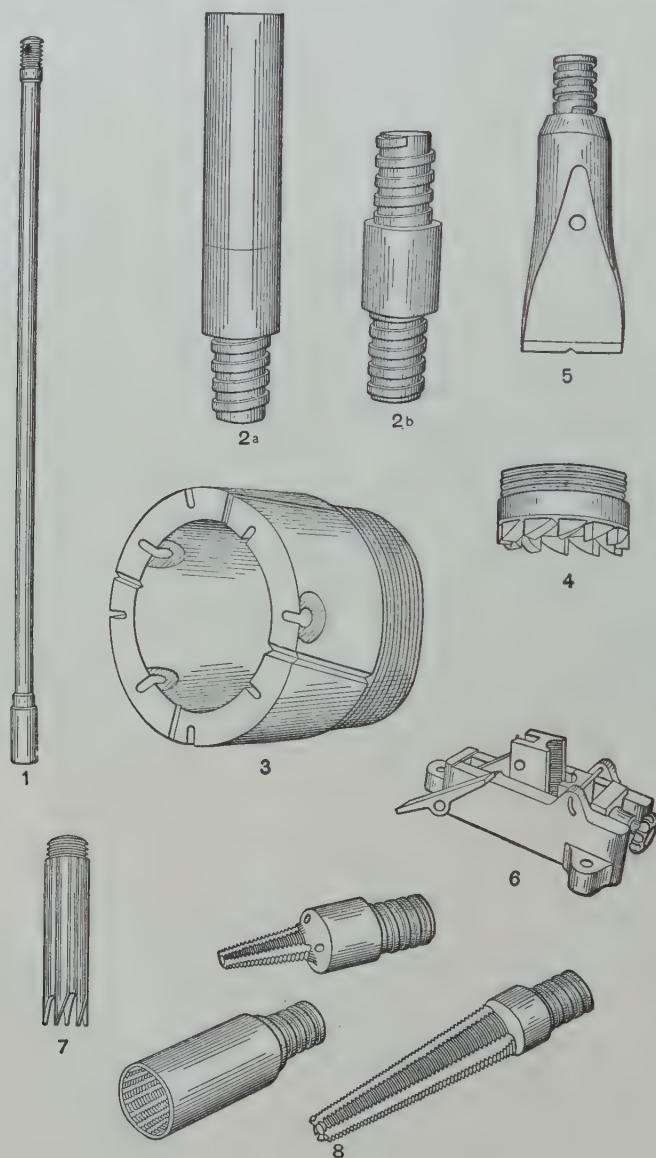


FIG. 99.—Diamond drill appliances—details.

drill rods the shot, which is of chilled steel about $\frac{1}{16}$ in. in diameter, bites into the rock and chips or wears off small pieces of it, which are washed to the surface.

Figure 100 shows a portable and convenient outfit as used for making shot-drill borings in explorations for a dam site in 1917, under the direction of the writer. The belt connects with the pulley of a small gasoline engine used for rotating the drill rod. The pipe derrick and hose connection may also be noted. Water was obtained through a line of 1-in. pipe by means of a small gasoline-operated pump, located at the bank of the stream.



FIG. 100.—Portable shot-drill outfit, in use at dam site.

The calyx drill [Fig. 99 (7)], or some modification of it made of chilled steel, may also be used for core borings where the rock is not too hard.

Comparison of Core-drilling Methods.—The diamond drill is better adapted than the shot drill where the rock is very hard but is difficult to use in flinty rocks or rocks of very uneven texture on account of the tendency to tear out the diamonds.

The diamond drill will bore at any angle, while the shot drill must be set to drill vertically. This is not usually of consequence, however, in explorations for dam sites.

The shot drill may be used to obtain larger cores than the diamond drill and is usually more economical for borings in softer rocks.

Cost of Borings.—The cost of borings will vary greatly, depending both upon the amount of work to be done and the character of material to be encountered. This is especially the case with core borings, where usually it is necessary to guarantee a certain minimum amount of work to cover the cost of furnishing the necessary plant and equipment. Hence, where only a small amount of core work actually has to be done, its cost per linear foot is high.

Assuming a fair-sized job, wash borings now cost from about \$1 to \$2 per linear foot; core borings from perhaps \$5 to \$8 per linear foot.

Frequently it is impossible to make reliable estimates of the cost of a dam without suitable data from core borings, and in any event more than the cost of borings may usually be saved in design and construction by the knowledge gained from them.

Location of Borings.—The number of borings required at a given dam site will vary greatly with the character of foundation material and the size and importance of the structure. Often, a single row of core borings on the center line of the dam spaced perhaps 50 ft. apart will be adequate, taking care that holes go into rock sufficiently to establish its character without question.

Very irregular or seamy rock may require intermediate holes; on the other hand, where the soil covering is not very deep, a test pit may suffice.

A high dam may require two or more lines of holes at least over part of its area; in any case it is well to have a few borings off the general line, both up and downstream, to show the slope of the rock surface.

The results of borings should be made available by record logs of each hole. Also, record drawings are usually required, with a vertical section of each boring (with designating number), constructed to some suitable datum and showing materials as determined from samples or as interpreted, at their proper elevation, with a key plan showing its location with reference to the proposed dam.

The section of each boring should be shown in its proper position upon a longitudinal center-line section through the dam site, and usually a line marked "surface of ledge rock" can be drawn, based upon the results of the borings. In case any borings are off the center-line section, this should be noted, or in the case of an important dam there may be two or more longitudinal sections.

TYPES OF DAMS

Requisites of a Dam.—The purpose of the dam in a water power development is primarily to afford a head of water, but its function in creating pondage or storage is also often of great importance.

The basic principles underlying the location and design of the dam, as well as of the other features of a water power development, are economy and safety. The dam is likely to be the most important feature of a power development and, therefore, the item of greatest cost, since more than any other part of the works it is exposed to the forces of nature, of water, and often ice pressure, which it must safely withstand at all times and under all conditions.

As will be further noted, there is often considerable uncertainty in the definition of, or allowance for, such forces as ice pressure and the amount of uplift, the latter particularly at the foundation of the dam. Consequently, theory and practice must be guided to a considerable extent by the results of construction and operation, as far as these afford positive information.

For safety, a dam must be relatively impervious, in both foundations and dam itself, and stable under all conditions. The term "relatively impervious" does not necessarily mean no leakage whatever past the dam but rather an amount not great enough to endanger in any way the stability of the dam or result in an undesirable waste of water. Ordinarily, this will mean, however, very slight leakage, if suitable design and construction are carried out, particularly in the case of masonry dams.

Masonry Dams.—Where a rock foundation is available, the solid masonry dam is the most permanent type of construction. At the present day such a dam would commonly be built of concrete, with hearting of cyclopean masonry, *viz.*, concrete with "plums" or large stones embedded. Formerly, many such dams were built of stone masonry, and occasionally today, under special circumstances, stone masonry might be used, although usually more expensive than concrete.

Hollow masonry dams are in quite general use and may be cheaper than solid dams where cement and concrete materials are high in cost. The Ambursen or buttress type of hollow dam, of which many have been built, is constructed of buttresses supporting a flat-deck slab of reinforced concrete. Arched decks between buttresses have also been built, without reinforcement. The hollow dam is also well adapted to use on foundations where uplift is likely to occur.

Arched masonry dams, *i.e.*, curved upstream in plan, located in a relatively narrow valley with steep slopes suitable for arch abutments, are of low cost and may be used where conditions permit. The multiple-arch dam is also an economical type of construction.

Earth Dams.—Earth dams may be classified as follows:

1. *With Core* (or central impervious section).—The core is usually of concrete but occasionally of clay puddle, commonly placed about in the middle of the earth section, which is trapezoidal with level top and suitable upstream and downstream slopes.

2. *Without Core*.—Unless of small height, earth dams without cores usually require that their material be selected and placed so as to render a portion of the dam cross section (usually the upstream third or half) relatively impervious. These may be called “rolled-layer” dams, the materials being commonly placed in layers and each layer rolled.

An effective method of grading materials, particularly where large quantities are involved, is hydraulic placing where material is hauled to the slopes of the dam (usually in cars) and washed into place with nozzle and jet, resulting in an impervious central portion of fine material. This is also sometimes called “semihydraulic” fill and is now in frequent use.

The hydraulic-fill dam differs from the foregoing in that materials are washed down in the pit from which they are obtained and carried hydraulically in flumes or pipes to the dam, thus transporting as well as placing hydraulically.

3. *The rock-fill dam*, as the name indicates, is a rough fill of loose rock, leakage through which is minimized usually by a deck or apron of wood, concrete, etc.

Wooden Dams.—Timber dams may be of log cribwork filled with rock, with plank deck or apron—a common type of construction in a new country—or of frame and deck construction for low dams.

Combinations of Types of Dams.—It will frequently be found advantageous to combine different types of dams at a given site. A very common arrangement consists of a concrete spillway or overfall section with earth wings at one or both ends of the dam, either with or without core wall. This is particularly useful in the very common situation at dam sites where ledge rock exists near the surface, perhaps in the river and on one bank, permitting a concrete spillway section here, but is at considerable depth on the other bank.

A hollow spillway section of dam with either concrete or earth abutments is also common.

FORCES ACTING ON MASONRY DAMS

The ordinary masonry dam is of “gravity section,” so called, in that it carries its load or loads by virtue of its weight. The forces acting on any section of the dam and its foundations, therefore, are those occasioned by the loads acting on the dam and its weight acting to resist these loads. Incidentally, stresses are occasioned in the dam and its foundations, which must be kept within proper limits.

The forces acting on masonry dams and stresses due to these forces will be discussed in the following order: (1) water pressure, (2) earth pressure, (3) ice pressure, (4) other forces, (5) weight of dam, and (6) stresses in masonry.

1. Water Pressure. *a. Abutment or Bulkhead Section.*—Referring to Fig. 101, it will usually be found more convenient to use components of water pressure so that for any height h and water at w lb. per cubic foot, $P_h = \frac{1}{2} wh^2$, acting at a depth of $\frac{2}{3} h$.

P_v = weight of 1-2-3-4 acting at the center of gravity of 1-2-3-4. For the distance from water surface to center of pressure of any submerged section as 4-3, this is conveniently expressed as $h_c = h_0 + \frac{d^2}{12h_0}$, where h_0 is the head on the center of gravity of 4-3 and d is the vertical projection of 4-3.

Forces due to backwater on the toe of the dam with head h_2 would be computed similarly to those at the heel.

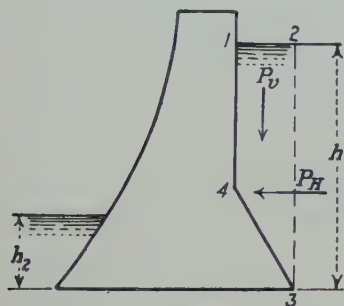


FIG. 101.

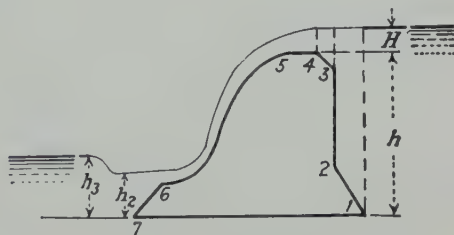


FIG. 102.

b. Spillway Section.—Referring to Fig. 102, vertical and horizontal components of water pressure on 1-2-3, or any portions thereof, will be determined as for the abutment section, keeping in mind the additional head H due to surcharge. It is common practice to neglect the vertical component of water pressure on 3-4-5-6 but to include the horizontal component on 3-4.

Backwater at the toe of the dam h_2 may be practically nothing, due to high velocity of flow over the spillway and small depth at h_2 , and the formation of an hydraulic jump, shown by h_3 . If the depth of water below the spillway is sufficient, the jump will be "drowned out" and backwater due to h_3 will occur.

The conditions under which the hydraulic jump will occur are clearly described in Part III of the *Technical Reports* of the Miami Conservancy District, pages 23-25. The inclusion of backwater at the toe of the dam is of most importance in respect to uplift, as further explained.

The force due to impact of flowing water will seldom be large in amount as compared with static pressures. For illustration, a spillway section 50 ft. high and 100 ft. long, with surcharge of 10 ft., will have the following:

$P_h = 50 \times 35 \times 62.5 = 109,000$ lb. per linear foot of static water pressure.

Discharge over the spillway per foot of crest would be about

$$3.5 \times 10^{3\frac{1}{2}} = 110 \text{ sec.-ft.}$$

and mean velocity of approach about 1.84 ft. per second.

$$\begin{aligned} \text{Hence, } P \text{ impact} &= \frac{W}{g} \Delta V \\ &= \frac{110 \times 62.5}{32.2} \times 1.84 = 392 \text{ lb. per linear foot} \end{aligned}$$

or an insignificant force. If the dam were 100 ft. high instead of 50 ft., this force of impact would become about 1170 lb. per linear foot of dam, as compared with 9400 lb. per linear foot of static pressure.

Actually, impact will be greater nearer the top of the dam than at greater depths, due to irregular distribution of velocity in the vertical section, with greater velocity nearer the top. Even allowing for this it will seldom be necessary to consider impact as one of the forces acting on the dam.

Upward water pressure or uplift may be a factor of importance, due to pervious masonry or foundations, or both.

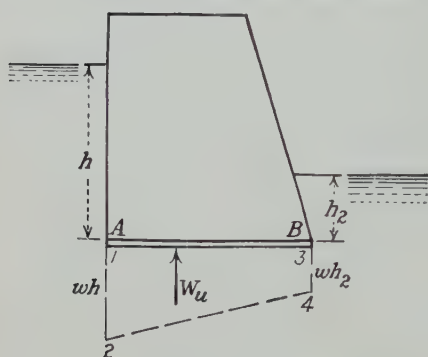


FIG. 103.

Referring to Fig. 103, if AB is a very small opening or passage

through a masonry dam with water loads as shown, it is evident that at A an upward pressure on the masonry will exist due to the head h , and at B due to head h_2 (neglecting any velocity head due to flow through AB). Now, considering the entire level of AB longitudinally through the dam, if such openings or passages occurred at frequent intervals, a considerable total upward water pressure might be exerted at this level. If the openings aggregated 50 per cent of the entire length of the dam at level AB , the upward force exerted would correspond to one-half the area of the stress diagram 1-2-3-4; if 30 per cent of openings, the total upward force would be 30 per cent of 1-2-3-4, etc. We may, therefore, express such uplift per linear foot of dam by the equation:

$$W_u = Cwl \frac{(h + h_2)}{2} \quad (1)$$

where C represents the proportion of effective uplift as regards area, and l the length of joint AB .

On such a level as AB , in a masonry dam there are no such continuous openings or passages between heel and toe of dam such as assumed,

even with relatively poor quality of masonry, and ordinarily the value of C in this formula would be small. In the design of the Wachusett, Ashokan, and Kensico dams, however, which were all high masonry dams used for water supply, a value of $C = \frac{2}{3}$ was taken.

The foregoing discussion considered the dam above its foundations. Evidently, uplift may occur on the foundation joint or in the foundations of a masonry dam, particularly where this foundation is of pervious rock, perhaps stratified as well. In fact, possible uplift on the dam as a whole, through pervious foundations, is the most important aspect of this subject.

It will be seen that the effect of uplift is to reduce the effective weight of the dam and, therefore, to diminish its ability to withstand sliding as well as increase any overturning tendency. Consequently, if uplift has to be assumed in the design of the dam, it will result in adding materially to the cross-sectional area of the dam and, hence, to its cost. It is better, however, and usually cheaper, as a rule, rather than to increase the section of the dam, to trench the foundation rock longitudinally near the heel of the dam, fill this with concrete masonry, and thus by a cut-off prevent upward water pressure from reaching the bottom of the dam. This may require a deep trench, if the foundation rock is very pervious or stratified, or even the grouting of the foundation on the line of the cut-off with cement mortar, as further described under foundations. Such a cut-off should effectively limit upward water pressure on the dam, but any water passing it should, by a suitable drainage system, be allowed to pass downstream and not build up pressure under the dam.

An excellent discussion of the subject of upward water pressure, and allowance therefor, was given by C. L. Harrison.¹

In the construction of the Sherman Island multiple-arch dam,² with foundation of sand and boulders requiring a cut-off of sheet-steel piling at the heel of the dam, seventeen 3-in. wrought-iron pipes were driven so as to extend about 10 ft. into the sand under the base slab and used to determine the hydraulic gradient after the dam was completed and the pond filled above the dam. It was found that the average loss of head, due to the sheet-piling cut-off, was about 38 ft. The loss of head in passing through the sand and boulder foundation was about 11 ft. in a distance of 150 ft., corresponding to an uplift of about 22 per cent of the 50-ft. net head between pond and tailwater levels. The original calculations included an allowance of 26 per cent acting on the entire base, but the actual uplift probably acts on less than 50 per cent of the area, due to sand contact. Another interesting result of the measurements at this dam was the relation between seasonal water temperature and uplift. It was found that, with water temperature varying from 65°

¹ *Trans. A.S.C.E.*, December, 1912, pp. 142-225.

² *Trans. A.S.C.E.*, vol. 88, pp. 1287-1289, 1925.

to 32°F., the uplift head varied from 11.3 to 7.2 ft. or about 1 ft. for each 8°F.

Experimental data upon uplift pressure at 10 dams studied by Houk¹ resulted in the suggestion that a tentative allowance for uplift would normally be as follows: At heel 100 per cent of total head; at 10 per cent toward toe, 50 per cent of head; at toe (without backwater), 0 per cent of head; with uniform variation in pressure between the above points. This would give an average uplift over the entire base of $0.30 H$, with resultant acting about 0.30 of base from heel of dam.

Further experimental data of this kind are desirable to aid the judgment of the engineer in allowing for the important factor of uplift.

2. Earth Pressure.—A dam may often be subjected to earth pressure due to filling to some level above the foundations. Such a fill on the upstream side of the dam, of course, will be saturated and diminished in weight accordingly. Thus, with earth which in the dry weighs 105 lb. per cubic foot, its weight under water, if one-third voids, will be $105 - 62.5 (1 - 0.33) = 63.5$ lb. per cubic foot, or not greatly different from the weight of water. Under such conditions, it would be assumed to act as a liquid weighing 63.5 lb. per cubic foot.

Where not liquid, if α is the angle of repose of the earth, earth pressure, *i.e.*, the horizontal component, would be computed by the Rankine formula,

$$P_e = \frac{W_e h^2}{2} \left(\frac{1 - \sin \alpha}{1 + \sin \alpha} \right)$$

where W_e is weight per cubic foot of earth, h is the depth in feet, and α its angle of repose. The vertical component of earth pressure on sloping faces of the dam will be the weight of the material vertically above the plane, just as for water pressure. In fact, the formula for horizontal earth pressure, as given above, is identical with that for water pressure, except as modified by the Rankine coefficient $\left(\frac{1 - \sin \alpha}{1 + \sin \alpha} \right)$. When α is 0 deg. (material practically liquid), this coefficient becomes unity; for an ordinary value of $\alpha = 30$ deg., it becomes $\frac{1}{3}$.

The weight of submerged earth is thus not greatly different from that of water, but its angle of repose and, hence, its effectiveness in producing a horizontal pressure against a dam vary from 0 to about 30 deg.

3. Ice Pressure.²—Ice pressure against a dam may be due to expansion of the sheet of ice in the pond or reservoir formed by the dam or to the formation of ice jams due to the ice in a river breaking up and going out quickly in time of sudden freshets. The latter is not, as a rule,

¹ HOUK and HENNY, "Uplift Pressure in Masonry Dams," *Civil Engineering*, A.S.C.E., September, 1932, pp. 578-582.

² BARNES, HOWARD T., "Ice Engineering."

as important in respect to loads imposed on the dam as from the point of view of causing excessive backwater and incidental damage at points upstream from the dam. Sheet-ice pressure, however, may exert a dangerously large force upon a dam under favorable conditions, due essentially to its expansion on account of rise in temperature.

Thus, where a complete sheet of ice covers a reservoir, a sudden cold spell and consequent contraction will crack the ice. These cracks will fill with water and freeze solidly, often over night. The next day rising temperature, especially when accompanied by bright sun, will cause the ice to expand (its linear coefficient of expansion as determined by Andrews¹ varying from about 0.00002 at -30°F. to 0.00004 at 32°F.), and if it cannot give way, either laterally by sliding up the banks of the reservoir or, if distance is not too great, on the reservoir bank upstream from, or opposite the dam, pressure on the dam may result in any amount up to that corresponding to the crushing strength of the ice. The latter factor, however, is quite variable, values being given all the way from 7 to 70 tons per square foot.

In the design of the Wachusett, Ashokan, and Kensico dams, previously referred to, it was assumed that clear block ice 1 ft. thick might be expected to form at the surface of the reservoir and expand so as to exert a crushing strength of 23.5 tons per linear foot (or per square foot expressed as unit pressure). In the design of the New Croton dam—the predecessor of the above three high dams—ice thrust was disregarded. For the upper St. Maurice River dam,² north of Montreal in Canada, ice pressure of 25 tons per linear foot was assumed.

Suitable allowance for ice pressure will evidently vary greatly with the location of the dam as to latitude, its arrangement with reference to the side shores of the reservoir, the slopes and condition of those shores, and the “reach,” or distance, upstream from the dam to the opposite shore. A reservoir normally well drawn down in winter time would offer little opportunity for ice pressure to be exerted at or near spillway level and, hence, no great stresses due to ice pressure. There is always the possibility, however, in such a case that unusual conditions of operation may result in a full reservoir at some time during the winter and an opportunity for maximum ice loading.

Harrison's paper previously referred to³ and its accompanying discussions review the subject of ice pressure and allowance therefor quite fully. Obviously, no general rule for such allowance can be made, each case being one to consider by itself. It is usually likely to be a case of either liberal allowance for ice pressure, or none whatever where the probability of occurrence is small.

¹ BARNES, HOWARD T., “Ice Formation,” pp. 64-65.

² *Eng. Record*, vol. 70, p. 394.

³ *Trans. A.S.C.E.*, December, 1912, p. 142.

Allowance of ice pressure, to such an extent as 23.5 tons per linear foot, results in the ice loading fixing the cross section of the dam in its upper levels or for dams of moderate height. In high dams the water loading is likely to fix the section of the lower portions of the dam, keeping in mind that maximum ice and maximum water loadings would practically never occur together.

4. Other Forces.—These may include *air pressure* on the spillway section, due to a partial vacuum forming under the nappe of water flowing over the dam due to imperfect aëration at the ends of the sheet of water. This results in unequal air pressure on the heel and toe faces of the dam and, hence, an additional force tending to push the dam downstream. This may not be great enough in amount to be troublesome, but strong vibrations are sometimes produced by periodic making and breaking of this partial vacuum. The remedy is so to arrange the ends of the dam or any piers that the formation of a vacuum under the sheet of falling water is prevented.

Wave pressure of itself is not usually of importance as a force to be withstood but may dangerously increase the head acting on the dam, particularly in the case of earth dams, which may not be safely overtopped with water.

Wind pressure is not of importance in dam design, as the maximum loading due to wind would always be small as compared to water loading, and these loads cannot occur simultaneously to their full extent.

5. Weight of Dam.—The weight of concrete masonry varies principally with the weight of its aggregate and somewhat with the proportions of the mix. Neglecting the effect of the latter and assuming an ordinary mix, the weight will be about as follows:¹

Stone	Weight of solid stone, pounds per cubic feet	Weight of concrete, pounds per cubic feet
Trap.....	180	155
Granite, etc.....	170	150
Limestone, etc.....	160	148
Sandstone, etc.....	150	143

In work of importance the weight of masonry may be estimated by determining the weight of aggregate used and proceeding as follows:²

Assuming trap rock at 180 lb. per cubic foot, 45 per cent voids when dust is removed; sand at 90 lb. per cubic foot dry; cement at 100 lb. per cubic foot; 1:3:6 concrete requiring 1.11 bbl. cement per cubic yard.

¹ TAYLOR and THOMPSON, "Concrete Costs," pp. 115, 132, 151, 159.

² *Ibid.*

Cement.....	$\frac{1.11}{27} \times 380 \text{ lb.}$	= 15.6
Sand.....	0.47 cu. ft. at 90 lb.	= 42.3
Stone.....	0.94 cu. ft. at [180 (1-0.45) = 100]	= 94.0
Weight per cubic foot		= 152 lb.

For cyclopean masonry, commonly used for all but the exterior portions of a masonry dam, such a 1:3:6 concrete as the above would have "plums" or large stones embedded in it, increasing the weight of concrete as follows, assuming, for example, 20 per cent of plums:

Concrete.....	$0.80 \times 152 = 122$
Plums.....	$0.20 \times 180 = 36$
Weight per cubic foot cyclopean masonry....	= 158 lb.

Concrete masonry as commonly used in dams, therefore, may weigh from about 140 to 160 lb. per cubic foot, depending on circumstances. Usually, for ordinary work or estimates, 150 lb. per cubic foot is suitable.

Stone masonry is not used commonly now for dams except in unusual circumstances, although it may be used for the facing of the dam as a more attractive finish, as in the case of the Kensico dam. In water power developments it would be seldom used, however, unless for special reasons stone masonry was less costly than concrete. The weight of stone masonry is usually somewhat greater than that of concrete. In the Wachusett dam, built of rubble-granite-masonry hearting and coursed-ashlar facing, the weight used in the design of the dam was 156.25 lb. per cubic foot (sp. gr. = 2.5).

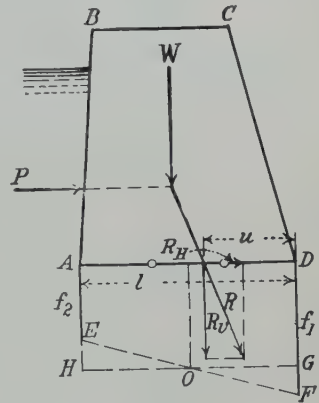


FIG. 104.

6. Stresses in Masonry. *a. Solid Dams.*—In Fig. 104, $ABCD$ represents a cross section of a solid-masonry dam assumed of unit length, above an assumed joint level AD , subjected to water loading P , with total weight of masonry above joint $AD = W$.

The resolution of P and W gives a total force R acting on the joint, with vertical component R_v causing compressive stress on the joint (diagrammatically shown as $AEFD$), and horizontal component R_h , which tends to make the section $ABCD$ slide on AD .

In considering pressures on a masonry joint, the material is assumed to be rigid and incapable of withstanding tension, although it is actually elastic to a certain degree and, in the case of good concrete masonry, can stand considerable tension. The exact law of stress variation,

therefore, is not known, but it is commonly assumed that the vertical stress in the joint varies uniformly, as in the design of beams. Hence, for the conditions shown, the stress diagram $A E F D$ will be a trapezoid made up of (1) direct stress $A H G D$, and (2) flexural stress $E H O G F$, and the stress at any point in the joint will be

$$f = \frac{R_v}{l} \pm \frac{R_v \left(\frac{l}{2} - u \right) \frac{l}{2}}{\frac{1}{12} l^3} \quad (2)$$

In computing such stresses the form of this equation $\left(f = \frac{P}{A} \pm \frac{M_v}{I} \right)$ should be kept in mind, rather than the particular formula resulting from its application.

If $u = l/3$ or the resultant force acts at the downstream third point of the dam, evidently f_1 will become double the average f , and f_2 will be zero.

If $u < l/3$, evidently there would tend to be tensile stress at the heel,

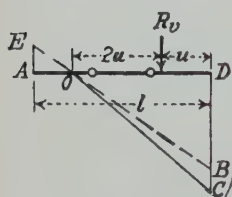


FIG. 105.

and a stress diagram as shown similar to $E A O B D$ (Fig. 105). As the joint is assumed to bear no tension, however, the stress diagram becomes the triangle $O C D$ (equal in area to $E A O B D$), and the maximum compressive stress $D C$ will evidently be $f_2 = 2R_v/3u$. The minimum stress at O is zero, while $A O$ carries no stress. (The foregoing also shows that, for no tendency to tension on the joint,

the resultant force acting on the joint must lie within its middle third.)

As R_v moves towards D or as u decreases, the maximum stress f_2 increases and theoretically would become infinite when $u = 0$ or when the resultant pressure passes through the joint limit. Practically, the compression at D would usually become so great before $u = 0$ that the masonry would crush, and overturning about some point inside of D would occur.

b. Hollow Dams.—The cross section of the ordinary hollow concrete dam is approximately of T-section instead of rectangular like the solid dam. Stresses may be computed by the same general method as outlined for solid dams, taking due account of the difference in section, etc. Thus, in Fig. 106, with the hollow dam, the resultant vertical force on a section is normally near the middle of the dam, at O . The center of gravity of the cross section of the dam will be at C , some distance e from this resultant force. The resulting stress at the toe will be

$$f_1 = \frac{R_v}{A} + \frac{R_v e m_1}{I} \quad (3)$$

and at the heel

$$f_2 = \frac{R_v}{A} - \frac{R_v e m_2}{I} \quad (4)$$

Here, again, the method rather than the specific formula should be kept in mind.

c. Inclined Stresses in Masonry Dams.—The vertical unit pressures or stresses that have been discussed are not the maximum intensities of pressure in the foundation or dam but rather the vertical components of such stresses. The intensity of “inclined stresses” may be determined as follows:

Consider a small element ADC at the toe of dam (Fig. 107), the weight of which is negligible. Shear on DC is zero and, therefore, on AD is also

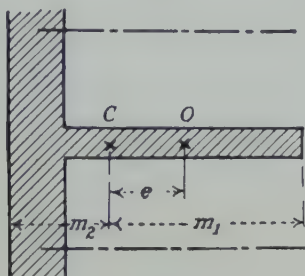


FIG. 106.

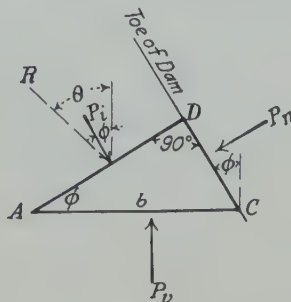


FIG. 107.

zero (because the intensities of shear on two planes at right angles to each other are equal), and stress on AD is wholly normal.

Total pressure on $AD = p_i \times b \cos \phi$.

If $\Sigma V = 0$, $p_v b = p_i b \cos \phi \cdot \cos \phi$, or

$$p_i = p_v \sec^2 \phi \quad (5)$$

ϕ varies from about 30 deg. for a low dam to 45 deg. for a high dam, for which limits $p_i = 1.33$ to $2.0 p_v$.

In case the angle Θ between the vertical and the direction of the resultant force acting on a joint (as AC) is $> \phi$, use $p_i = p_v \sec^2 \Theta$.

If an additional normal force p_n exists at the toe due to either water or earth pressure:

$$\begin{aligned} p_v b &= p_i b \cos^2 \phi + p_n b \sin^2 \phi \\ \text{or} \quad p_i &= p_v \sec^2 \phi - p_n \tan^2 \phi \end{aligned} \quad (6)$$

REQUIREMENTS FOR STABILITY OF MASONRY DAMS

A masonry dam of gravity section, whether of spillway or abutment type, must be able to withstand with suitable factor of safety: (1) overturning, a tendency to overturn about the downstream edge of the dam, at any assumed joint level in the dam, including the base of the dam and its foundation; (2) sliding, a tendency to slide on any assumed horizontal joint in the dam or its foundation; (3) stresses, occasioned in

the masonry of the dam by its loading; (4) other possible stresses or conditions requiring special design. These requirements will be discussed in detail.

1. Overturning.—Theoretically, overturning would take place about the toe of the dam if the resultant forces acting at any joint level should come outside the joint at the toe. It is not desirable, however, to have any tendency to tension at the heel of the dam, and, hence, the rule is commonly adopted of requiring the line of resistance to lie within the outer third point of the dam, which incidentally results in safety as regards overturning.

In a high dam there is also the possibility of the line of resistance coming upstream from the inner third point, when the reservoir is empty, and, hence, a tendency to tension at the toe of the dam under these conditions. This requires usually that the heel of the dam be sloping outwards, below a certain level.

2. Sliding.—The tendency to slide is caused by the horizontal component of the resultant force acting on any given joint level. It is withstood by the weight of the dam (increased by any effective vertical components of earth or water pressure), effectively limited by the coefficient of friction of the materials forming actual or assumed joints in the dam or its foundations. This may be of concrete on concrete, for example, or concrete on rock (bottom of the dam), or rock on rock (in the foundation).

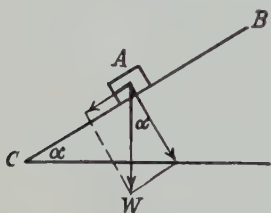


FIG. 108.

Referring to Fig. 108, if the block of concrete at *A*, which weighs *W* lb., is just on the point of sliding down the concrete slope *BC*, if *f* is the coefficient of friction:

$$W \sin \alpha = W \cos \alpha \times f$$

$$f = \tan \alpha$$

or

In other words, considering *BC* as a masonry joint, *W* will be the resultant force acting on the joint, making an angle α with the normal to the joint, and the coefficient of friction is the tangent of this angle α .

Hence, just to prevent sliding on a horizontal masonry joint the resultant force acting on this joint must not exceed an angle with the vertical whose tangent is the coefficient of friction under the given circumstances.

Ultimate values of *f* for concrete on concrete and concrete on rock, as in masonry dams, are not known with exactness but are probably in the vicinity of 0.8. It is customary to limit this value of *f* or tangent with the vertical for ordinary rock foundations to about 0.5 to 0.55, which will provide some factor of safety against sliding.

Of course, a masonry dam founded on earth would require a much lower value of f , perhaps 0.3 to 0.4, depending on its character. Such construction is usually undesirable, however, as previously noted.

In addition to limiting the angle of the resultant with the vertical, it is also desirable to avoid horizontal joint planes in the concrete between successive days' work in pouring, by care in joining old and new concrete and by suitable "keying" of the concrete by wooden blocks to form grooves in the old concrete, as well as by the use of plums lying partly above and partly below the joint level. The foundation of the dam should also be roughened, if necessary, and even excavated to give suitable "keying" of concrete as further described (see page 331).

3. Stresses.—For dams of ordinary height, say, up to 75 or 100 ft., the necessary cross section will usually be determined by one of the foregoing provisions, *viz.*, to keep the line of resistance inside the middle third or to be adequate as to sliding. In the case of a high dam, stresses in the lower part of the dam are likely to limit or fix its cross section at these levels.

The ultimate strength of stone masonry is not known from tests but is probably 150 tons per square foot or more. Foundation ledge rock would much exceed this in strength. Unless, therefore, foundation conditions are poor, with relatively soft rock, pressures in the masonry of the dam rather than on the foundation will limit in respect to compressive stresses.

The ultimate compressive strength of concrete varies with its age and mix. The following is compiled from Taylor and Thompson's "Concrete—Plain and Reinforced," based upon compressive tests of cube specimens, with 45 per cent of voids in stone:

TABLE 63.—CRUSHING STRENGTH OF CONCRETE—TONS PER SQUARE FOOT

Proportions	Age	
	One month	Six months
1-2 -4	176	237
1-2½-4	163	220
1-3 -5	147	197
1-3 -6	140	189

Stresses as used in the design of some of the high masonry dams previously mentioned are as shown in Table 64.

As will be noted from this table, an allowable maximum vertical stress of at least 15 tons per square foot is common, keeping in mind that this will normally be limited to a comparatively short length of dam, supported somewhat laterally by foundation rock at the sides of

the dam. This will usually keep the inclined or maximum stresses inside of 20 tons per square foot or roundly a factor of safety of about 10.

For a very high dam—in excess of 300 ft.—these stresses may probably be somewhat exceeded with safety; vertical stresses, say, 18 to 20 tons per square foot at the toe and 25 to 30 tons per square foot at the heel (reservoir empty). A study for such a dam as made by W. C. Sadler in 1921, as a thesis study at Massachusetts Institute of Technology, is shown in Fig. 109.

TABLE 64.—STRESSES IN HIGH MASONRY DAMS AS DESIGNED AND BUILT—TONS PER SQUARE FOOT

Dam	Date completed	Maximum height, feet	Vertical stresses		Inclined stresses	
			Toe	Heel	Toe	Heel
New Croton.....	1906	300	16.7	15.4	18.5	15.8
Wachusett.....	1905	205	10.0	12.0	10.7	
Ashokan.....	1914	220	8.7	14.7		
Kensico.....	1917	300	15.5	16.0	21.8	17.6
Arrowrock.....	1915	349	23.6	24.5	38.4	24.5
Elephant Butte.....	1916	301	14.2	17.3	28.3	17.4
Owyhee.....	1932	417	16.5	27.3	23.0	27.3
Hoover.....		730	15.6	39.2	25.0	45.0

4. Other Possible Stresses or Conditions Requiring Special Design.

Care must be taken in designing a high masonry dam not to give the toe of the dam too great a slope or angle with the vertical. If carried to the extreme, this would result in tension in vertical planes near the toe and a possible tendency to cracking off of masonry on these planes. If the toe slope is made not flatter than about 1:1 this condition cannot occur, and it will be noted that all of the high dams listed in previous tables have toe slopes approximately equal to this angle.

It will often be necessary to place a gate house, with wells or gate chambers, in the dam, and in designing such portions of the dam care must be taken not to weaken the dam in this section to any great extent. This may usually be done by increasing the width of the dam at heel or toe, or both, in the vicinity of the gate house.

Naturally, all factors of safety in design will be chosen, keeping in mind the importance of the structure and the possibilities of damage and loss of life in case it should fail. A dam of moderate height in a country of sparse population will justify greater boldness in design than such a structure as the Wachusett dam, which is of great height and is located just upstream from a good-sized city.

DESIGN OF MASONRY DAMS—ABUTMENT SECTION

The design of a masonry dam may be made most conveniently by assuming a tentative cross section based on the theoretical considerations which have been discussed, modified to meet other practical considerations, most of which will be obvious. While this procedure may be by analytical methods and by the solution of equations to give joint lengths, framed to express the conditions imposed by the various requirements

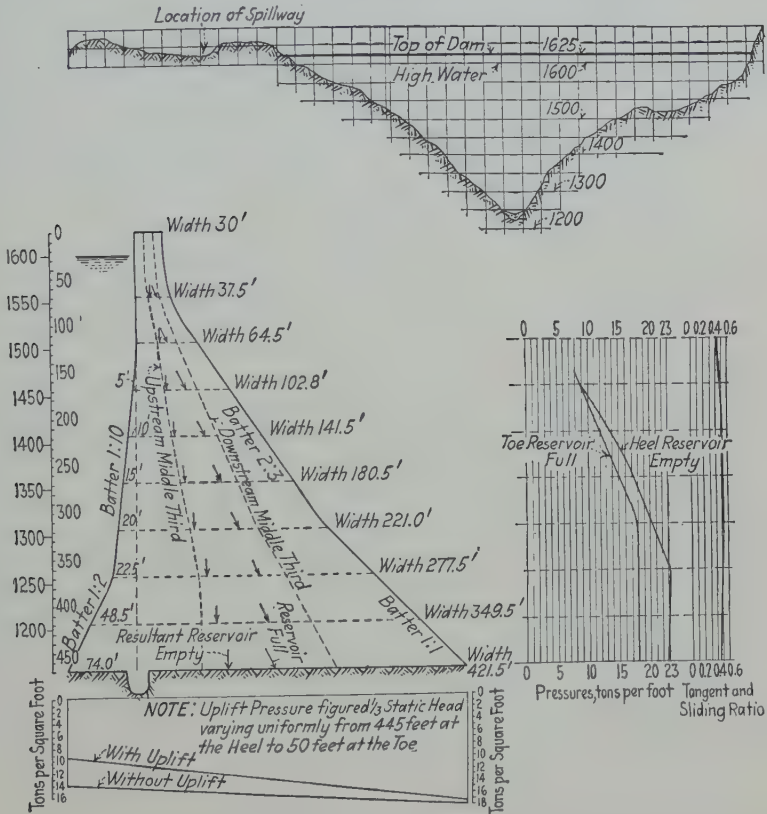


FIG. 109.—Design for a high masonry dam. (W. C. Sadler.)

which have been discussed, such equations are long and complicated and the method of trial of a tentative section using a combination of graphical and analytical methods is preferable. It is essential, furthermore, to keep in mind the uncertain assumptions which have to be made for certain loadings, notably ice pressure and uplift, making it unnecessary to fix joint lengths or the width of dam at any given elevation with great precision.

While the general methods of testing the tentative section of a masonry dam are the same for abutment and spillway type, the basis

for the section itself and its form varies somewhat for the two types. The base section for the abutment type will be first considered and the steps outlined for its design followed by similar procedure for the spillway section.

Basis for Section of Abutment Type.—Considering the three sections (Fig. 110) shown of height h subjected to water loading, by taking moments it will be found that, in order that the resultant of the forces acting on the base b shall just lie at the outer third point A , the necessary width of base b will be as follows (where w and y are unit weights of water and masonry, respectively):

$$(1) \ b = \sqrt{\frac{w}{y}}h; \quad (2) \ b = h; \quad (3) \ b = \sqrt{\frac{w}{y}}h$$

or for ordinary values of $w = 62.5$ and $y = 150$ lb. per cubic foot

$$(1) \ b = 0.65h; \quad (2) \ b = h; \quad (3) \ b = 0.65h$$

As the ratio w/y is always < 1 , section (1) would be more economical than (2) and have only half the material required for (3). The base section for a solid masonry dam not intended to be overtopped, therefore, is a right triangle with hypotenuse downstream. It is also of interest to note that the right triangle with hypotenuse upstream is approximately the base section for the hollow concrete dam, the ordinary slope with the vertical being about 1 on 1 instead of 1 on 0.67, as deduced above for the solid dam. (The downstream face of the hollow dam, however, commonly slopes about 15 deg. with the vertical.) The rectangular section is clearly not economical for either solid or hollow dam.

Considering the base section for the solid dam, with $b = 0.65h$, in respect to stability against sliding:

$$\frac{0.65h^2}{2} \cdot y \cdot f = \frac{1}{2}wh^2$$

or

$$f = \frac{w}{y \times 0.65} = 0.64$$

for ordinary values of w and y . This is a little high as a safe value for f . Taking the latter at 0.55 would require $b = 0.75h$. On the other hand, modifications in the practical section to give the dam some height above water level and some thickness at and above water level will add to the weight of the section. Hence, a base about $0.7h$ will ordinarily be sufficient.

Practical Section—Abutment Type of Dam.—The top width 1-2 (Fig. 111) of a solid masonry dam is usually from about $0.15h$ for low dams to $0.1h$ for high dams. The top of the dam is also about $0.15h$ to $0.1h$ above ordinary water (or spillway) level, although this dimension 1-3 will vary somewhat with particular conditions, such as spillway length, flood discharge over spillway, length of reservoir reach as affecting height of waves, etc. The toe face of the dam will be curved somewhat as shown by 4-5.

The above section will ordinarily be proper to use as a trial section in design. Depending on the height of the dam, the section will be found in design to be modified somewhat, and in general it will be noted that the width of the dam in its upper part is fixed by practical considerations; lower down, the width is fixed by the requirement of line of resistance lying within the middle third; still lower down, by limitation of allowable stresses in the masonry.

Method of Design—Abutment Section.—To test the assumed section and as a basis for its modification, a combination of graphical and analytical methods will be found convenient. Two such methods are shown in Fig. 112.

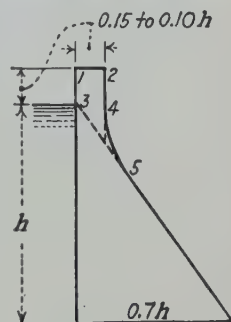


FIG. 111.

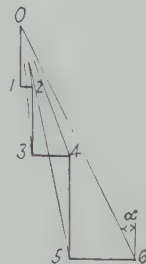
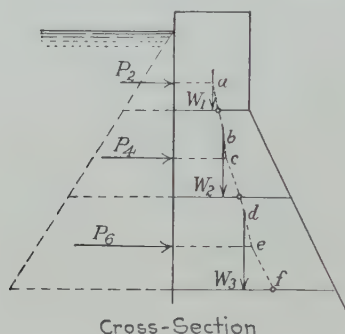
In either case the tentative section is subdivided by horizontal joints assumed for convenience every 10 ft. or so, depending on the height of dam under consideration. By the first method shown in Fig. 112, each subsection is considered by itself, the water loading P_2, P_4 , etc., and the weight of masonry W_1, W_2 , etc., computed in magnitude and location. Values of W and their location may be conveniently obtained by dividing the subsection into rectangles and triangles, scaling distances, and by taking moments about either heel or toe of joint, computing the distance to the center of gravity of the area. By a force polygon, values of P and W are combined and the location found of the line of resistance for each joint level. If this comes too far away from the third part of the joint for any subsection, the section of the dam above that level must be revised accordingly. The allowable limit in $\tan \alpha$ of 0.5 to 0.55 must also be kept in mind, and, as sections lower down are considered, it will be necessary also to compute stresses at toe and heel of dam (the latter for reservoir empty).

By the second method shown in Fig. 112, the total forces acting above an assumed joint level are computed and used instead of those for each subsection. This simplifies the determination of the point of application of the water loads P_2, P_4 , etc., and, on the whole, is the more convenient method.

In case other loadings are considered, as, for illustration, ice pressure or upward water pressure, this will simply add to the forces under consideration, making a somewhat more complicated procedure; the general steps in investigating their effect, however, will be the same.

Basis for Section—Spillway Type of Dam.—The spillway section of a dam must be designed to carry over it safely a sheet of water which may vary from a small depth to that for the maximum flood discharge. The

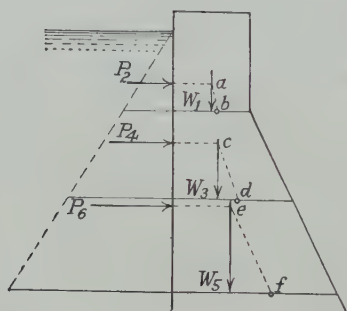
1. Using forces acting on each assumed section



Line	Parallel to
a-b	0-2
b-c	0-3
c-d	0-4
d-e	0-5
e-f	0-6

Force Polygon

2. Using sum of forces above a given assumed section



Line	Parallel to
a-b	1-2
c-d	3-4
e-f	5-6

FIG. 112.

length of spillway is usually fixed so as to limit approximately the maximum possible head on its crest. Design of the spillway section must be based on this maximum water load, and in some cases an ice loading at about spillway level must also be assumed, although these loadings would never be assumed simultaneously.

Another important consideration not occurring with the abutment section must be kept in mind in designing the upper portion of the spillway section, *viz.*, to so proportion the curve of the upper part of the spillway that the overfalling sheet of water will not leave it but rather be

guided steadily to the lower level and by a reverse curve started horizontally downstream again without undue shock or wearing on the toe of the dam or its foundations.

The theoretical path of particles of water going over the dam will evidently vary with their initial velocity, which will be greatest for the maximum head on the crest.

The theoretical or base section for the spillway will be a trapezoid, as shown in Fig. 113, which is the portion of a triangle of height $(h + H)$ and base about $0.7(h + H)$.

It will be found that this section will satisfy the rules relating to position of resultant inside the middle third and that for sliding.

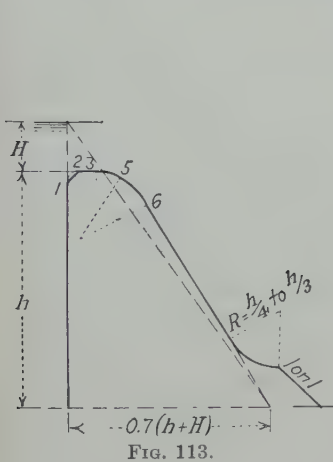


FIG. 113.

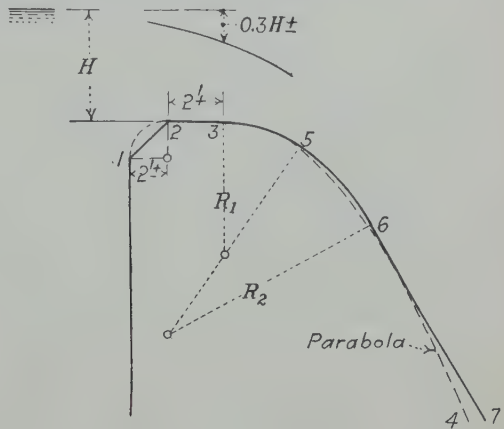


FIG. 114.

This base section will be modified at its top or crest as shown in Fig. 114.

The slope 1-2 (or curve as shown) will eliminate contraction of the flowing water and allow floating material—ice, logs, etc.—to pass the crest more readily without injuring the latter. The horizontal section 2-3 should give room for placing and removing flashboards and enough width for safety in walking along the crest.

The curve 3-5-6, with radii R_1 and R_2 , approximates in form and lies a little outside of the parabola 3-4, which is the locus of a particle of water leaving 3 with a certain assumed velocity, as further explained. Below 6, the face of the spillway would usually have a constant slope (unless of considerable height) to the elevation where the reverse curve begins (see Fig. 113). This curve would usually have a radius of $h/4$ to $h/3$ and terminate at the horizontal. If the latter point is above the foundation level, the latter would be reached by a 1-on-1 slope as shown.

The parabola for determining the curves to be used near the top of the spillway may be determined as follows: The head H on the spillway

enables the discharge per linear foot Q_1 of crest of spillway to be computed based on a suitable value of C in $Q = CLH^{3/2}$. At the flat portion of the spillway crest, near point 3 in previous sketch, the depth of flowing water is usually somewhere between 0.67 and 0.75 of H . Frizell¹ has shown that, theoretically, the drop in the surface curve is $H/3$. Practically, it is usually less than this amount, and a reasonable assumption is about $0.3H$.

With the water area known or assumed at 3, the mean velocity past 3 may be computed. Then, if this is v_m , and x and y are coordinates of the parabola at any point,

$$x = v_m t; \quad y = \frac{1}{2}gt^2$$

whence $y = \frac{1}{2}g \frac{x^2}{v_m^2}$, the equation of the parabola.

The velocity of a particle of water at point 3 is less than v_m , the latter lying usually about one-third of the depth upward from point 3. The exact value of the theoretical mean velocity at any point on the parabola may be computed by noting that the velocity v_p at any point on the parabola is $\sqrt{(v_m)^2 + (gt)^2}$. The area (or width of a section of unit length) of the sheet of falling water W_p at any point will be Q/v_p , and plotting $W_p/3$ normal to the parabola toward the spillway face will give a point on the lower nappe of the falling sheet of water to be used in fixing the curve of masonry of the spillway face. It will be found, however, that substantially the same result with materially less computation will be obtained by assuming v_m to act at point 3 and constructing a parabola beginning at point 3. Instead of using circular curves at 3-5-6, the parabolic curve may be used, defining this by its equation. For illustration, on the Bartlett's Ferry dam (see Fig. 222) this curvature was defined by the equation $x^2 = 50y$. Note also in Fig. 222 the overhang at the heel of the dam, permitting some economy of materials.

More experimental data are needed on the form of curve of the overfalling sheet of water for spillways and the amount of drop in this surface curve at the downstream limit of the flat portion of the crest. This was determined for the Garvin's Falls dam on the Merrimack River in 1923² for a head of 5.8 ft. on the crest as approximately 1.6 ft. or a drop in the surface curve of about $0.28H$. In 1926,³ with a head of 3.65 ft., the drop was 1.05 ft. or about $0.29H$.

In constructing the parabola used as a basis for the spillway face, a discharge somewhat less than the maximum expected may be used, as the former is usually of very rare occurrence and would prevail for only a short time. Backwater on the toe of the spillway at time of maximum

¹ "Water Power Engineering," p. 199.

² GILMAN and WENZ, Thesis, Massachusetts Institute of Technology.

³ PETERSON, G. R. and W. E., Thesis, Massachusetts Institute of Technology.

flood, furthermore, would usually help considerably to lessen the shock of the falling sheet of water.

Approximate Estimates of Masonry for Solid Dams.—For approximate preliminary estimates of quantities of masonry, the tentative sections of the masonry dam may be used to develop formulas as follows for solid dams up to about 100 ft. high.

Abutment Section.—Assuming a base = $0.7h$, and allowing for additional masonry at the top about as previously described, the cross-sectional area will be

$$\begin{aligned}\text{Area} &= \frac{0.7h^2}{2} + 0.3h \times 0.15h \\ &= 0.4h^2\end{aligned}$$

Cubic yards per linear foot Y_A will be

$$Y_A = \frac{0.4h^2}{27} = 0.015h^2 = \frac{h^2}{66} \quad (7)$$

Spillway Section.—With base = $0.7(H + h)$, top width $0.7H$, and adding 20 per cent for bucket area and additional area required by the parabolic curve of spillway face:

$$\text{Area} = 1.2 \times \frac{0.7}{2}(H + H + h) \times h = 0.42(h^2 + 2Hh)$$

Cubic yards per linear foot Y_s will be

$$Y_s = 0.015(h^2 + 2Hh) = \frac{h^2 + 2Hh}{66} \quad (8)$$

These formulas give approximately the masonry required for practical sections of any height h up to, say, about 100 ft., including depth of foundations. Thus, for heights of 40 and 80 ft. and $H = 10$, we should have

Abutment section:

$$\begin{aligned}h = 40: \quad Y_A &= \frac{1600}{66} = 24 \\ h = 80: \quad Y_A &= \frac{6400}{66} = 97\end{aligned}$$

Spillway section:

$$\begin{aligned}h = 40: \quad Y_s &= \frac{1600 + 800}{66} = 36 \\ h = 80: \quad Y_s &= \frac{6400 + 1600}{66} = 120\end{aligned}$$

These results will be found to accord fairly well with actual dam sections of these heights.

HOLLOW DAMS¹

Many hollow dams of the Ambursen, or slab-and-buttress, type have been built and are in use. The detailed features of this type of dam are shown in Figs. 115 and 116, the Rommel dam of the Arkansas Light and Power Company. It consists of a slab, or deck, of reinforced concrete supported by concrete buttresses, which, on foundation material other than rock, may require concrete footings, with sometimes a continuous mat of reinforced concrete between these footings.

The cross section of this type of dam is usually triangular, with upstream face or deck at an angle of about 45 deg. with the vertical.

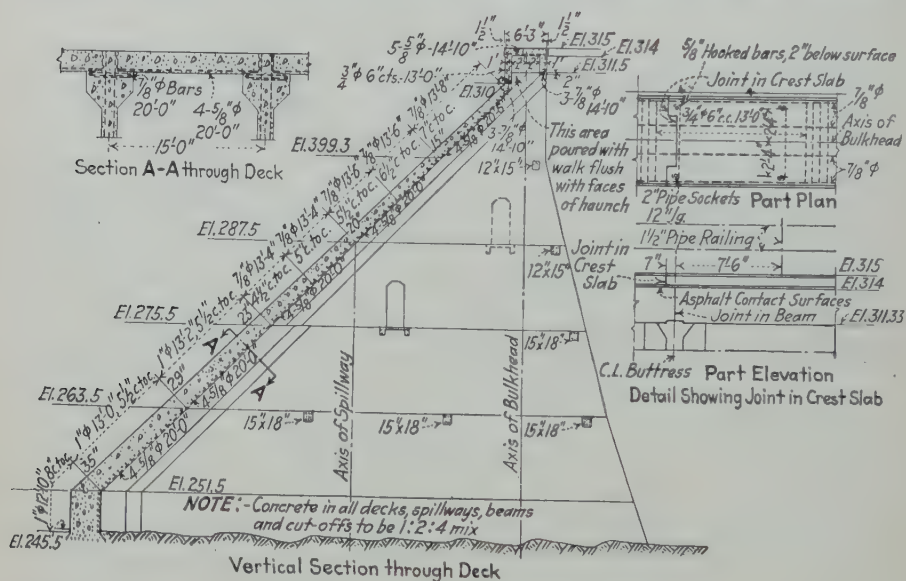


FIG. 115.—Rommel dam—bulkhead section.

The plane through the downstream ends of the buttresses for the abutment section of dam usually makes an angle with the vertical of about 15 deg., although for a low dam this may be vertical. It may also be desirable to give the dam some width at the top to permit walking or driving across it, thus somewhat modifying the triangular cross section. For the spillway section of dam, the downstream face must follow a suitable curve to fit the falling sheet of water, as explained under Solid Dams, and requires a substantial reinforced deck to carry water, ice loadings, etc.

The general method of design is to adopt a tentative section based upon previous designs and test this by computation, adjusting it as necessary.

¹ The author is indebted to the Ambursen Construction Company of New York City for much of the material used in the treatment of this subject.

Sliding.—The tangent of the angle Θ , which R makes with the vertical (Fig. 117), or the necessary value of coefficient of friction to prevent sliding, will be

$$f = \tan \Theta = \frac{0.31}{0.55} = 0.56$$

If the full range in proportion of solid concrete used for the hollow dam $\frac{1}{5}$ to $\frac{1}{3}$ is considered, the value of f would vary from about 0.62 to 0.49, as compared to the standard previously suggested of 0.55 to 0.50 for solid dams. With a solidity ratio of $\frac{1}{4}$ or better, the requirements for sliding, therefore, are met.

On foundation material other than rock, furthermore, a sliding coefficient of 0.4 to 0.5 is obtained by flattening the deck angle and using a floor between buttresses.

The hollow dam has the advantage over the solid dam in that uplift may be practically eliminated by providing suitable weep holes in the mat of concrete between buttresses, so that its actual stability against sliding may often be as good as or better than that of the solid dam.

Stress in Masonry.—Stresses at toe and heel of buttresses must be limited to proper values by the method previously explained (see page 292). Also the reaction of deck on the buttresses at the heel of the dam must be kept down to 15 or 20 tons per square foot for vertical stresses, or probably not over about 25 tons per square foot for inclined stresses.

Practical Details of Design.—The spacing of buttresses varies with the height of the dam. The usual spacing on rock foundations is 20 ft. on centers for heights up to about 100 ft. A spacing of 30 ft. may also be used up to heights of about 150 ft. where the overburden can be trenched to save excavation.

On soft foundations buttresses are spaced from 20 to 30 ft., a reinforced-concrete mat is used for the base and the slopes of the dam made flatter to obtain better sliding resistance.

From 150 to 500 ft. height massive buttresses spaced from 40 to 60 ft. on centers are used.

The buttresses, if high, should also be braced with rectangular reinforced-concrete struts, with additional horizontal steel in the buttresses at the level of each strut. The buttresses are braced by the rabbeted decks, where they join the upstream buttress tongue. Also, when the dam has a downstream apron, it is customary to drop the underside of the apron so that it has a 1-in. bearing on each side of each buttress.

The minimum thickness of buttress at top should be 12–15 in.; at the bottom it will depend on the height and stress allowed and may be 6 ft. or more for a high dam. For high dams buttresses are sometimes designed with pilasters and webs, which materially reduce the necessary yardage of concrete.

The upstream slab or deck at any given depth is designed for a uniform load of water pressure, assuming the slabs as separate between buttresses, or a moment of $Wl/8$ (W including the water load and the normal component of the weight of the deck slab). The slabs are also sometimes arched between buttresses to give strength in case the reinforcement deteriorates with age.

The proper thickness of slab may be quickly approximated by means of the formula

$$M = 108bd^2$$

where $M = Wl/8$, the moment in inch-pounds for a strip of width b (usually taken as 1 ft.), and thickness d , between type of slab and steel. This is assuming steel stressed to 16,000 lb. per square inch in tension and concrete 650 lb. per square inch in compression, with a ratio of area of steel to concrete of 0.0077.

In the cylinder or round-head buttress type of hollow dam as recently developed by Noetzli¹ the upstream end of the buttress is expanded to a cylindrical surface that extends laterally to meet the similar surfaces on the adjacent buttresses, and the water load is transmitted radially to the water-bearing surface, eliminating bending and diagonal tension. Sections are massive and no reinforcing steel is required.²

Spillway Section of Hollow Dam.—The spillway crest and apron must conform approximately to the sheet of water passing over the dam, with a suitable "bucket" or return to the horizontal at the toe. The thickness of the spillway apron cannot be accurately computed owing to its indeterminate loading, but will need to be somewhat thicker at the crest than on the sloping portion. Details of the spillway section of the Rammel dam are given in Fig. 116.

Relative Cost and Life of Hollow and Solid Dams.—The quantity of concrete in a hollow dam is of course much less in yardage than for a solid dam of the same height. For the bulkhead section, this ratio of yardage required will be about as 1:3; for the spillway section, with bucket, about as 1:2½.

The hollow dam, however, must be of reinforced concrete, which with the steel required will cost, under ordinary conditions, at least twice as much per cubic yard as the plain or cyclopean concrete in the solid dam. This would make the cost of the solid and hollow dams about in the ratio of 1½:1 for bulkhead section, and 1¼:1 for spillway section. Hence, there is ordinarily a margin in cost favoring the hollow dam, which actually may be from 10 to 30 per cent, depending on conditions. Where concrete materials are unusually costly, the hollow dam will show a better

¹ *Trans. A.S.C.E.* vol. 96, pp. 692-695, 1932.

² "Recent Advances in Buttress-type Dams," *Civil Engineering, A.S.C.E.*, February, 1931, pp. 387-391.

margin of economy; and, on the other hand, where concrete materials are cheap and form work costly, the solid dam may be cheaper.

As compared with an earth-fill dam, the hollow dam may be more economical, especially where the former has to have a masonry core wall. There are so many factors affecting the comparison, however, that each case must be considered by itself.

The hollow dam usually takes less time to construct than either the solid concrete or earth dams, due to the lesser quantity of materials to be placed, an element in its favor often of importance.

The life of the reinforced-concrete dam has not yet been fully determined from actual results, although there are numerous examples of such dams, built during the last 25 years, which are in excellent condition. Good workmanship and materials are essential with any type of dam, and, with the usual covering of the steel reinforcement by concrete, deterioration of steel should not occur.

The hollow dam is likely to be more permanent, however, in localities free from severe weather conditions, as its relatively thinner sections are subjected to greater frost and temperature effects than would be the case with the solid dam. Even this disadvantage may be avoided largely, however, with the bulkhead section of the hollow dam by constructing a face or curtain wall at the downstream side of the dam, thus creating a dead air space inside the dam.

The economy of the hollow dam and other advantages that have been enumerated, therefore, have resulted in much use of this type of dam.

ARCH DAMS

In a relatively narrow canyon or valley with steep sides where rock outcrop prevails, it is often possible to use a masonry dam curved upstream in plan so that stresses may be carried by arch action to the natural rock abutments. Where sufficient curvature can be obtained, the arch dam will be found to require much less masonry than a solid gravity dam for a given height.

Required Section.—Assuming arch action alone, the necessary thickness of dam for any depth of water h may be determined as follows: Referring to Fig. 118, $ABCD$ represents a horizontal section of unit height of an arched dam of thickness t , subjected to water pressure due to head h on the upstream face AB , which is of radius r . Considering unit areas at points N_1 and N_2 , which are symmetrical with reference to the center line OO' , and resolving the equal normal unit pressures P_1 and P_2 into components H_1 and H_2 normal to, and V_1 and V_2 parallel to, the axis OO' , it is evident that H_1 and H_2 balance, and the effect of P_1 and P_2 on AB is represented by $V_1 + V_2$. This will be true for all unit areas symmetrical with reference to OO' , so that the total force acting on AB will be $\Sigma(V_1 + V_2 + \text{etc.})$ or ΣV , which is $wh \cdot 2C$ where C is half the

chord AB . Each abutment, therefore, must carry a normal or arch stress R , which is

$$R = \frac{wh \cdot 2C}{2} \times \frac{r}{C} = whr$$

If allowable arch compressive stress is f , the necessary thickness will be

$$t = \frac{whr}{f} \quad (9)$$

which is identical with the basic formula for thickness of pipes and cylindrical containers, which, however, are usually subjected to internal pressure, causing tension in the pipe ring rather than compression.

This formula will evidently give a triangular cross section of dam with vertex at water surface, which must be modified for the practical section by assuming a suitable height of dam above water level and a suitable thickness at the top. These dimensions will vary somewhat with circumstances, but each may be taken at approximately $h/10$, with downstream face vertical to the elevation of water level and thence sloping uniformly to a width at base of dam as determined by the previous formula. This will give some excess of material, principally in the upper half of the dam, over that required by the previous formula.

Masonry required for this section in cubic yards per linear foot will be approximately:

$$\begin{aligned} Y_{\text{arch}} &= \left(\frac{whr}{f} \times \frac{h}{2} + \frac{1.1h \times 0.1h}{2} \right) \frac{1}{27} \\ &= \frac{h^2}{27} \left(\frac{wr}{2f} + 0.055 \right) \end{aligned} \quad (10)$$

Assuming $w = 62.5$ lb. per cubic foot and $f = 50,000$ lb. per square foot (a suitable and ordinary value for ordinary heights), this reduces to

$$Y_{\text{arch}} = h^2(0.0000231r + 0.00204) \quad (11)$$

Limit in Radius of Curvature.—Evidently, there will be a limiting value of r , say r_e , where the masonry required per linear foot of dam will be the same as that for the gravity section, determined as follows:

If $h^2(0.0000231r_e + 0.00204) = h^2/66$ (see page 303),

$$\begin{aligned} 0.00152r_e + 0.135 &= 1 \\ r_e &= 570 \text{ ft.} \end{aligned}$$

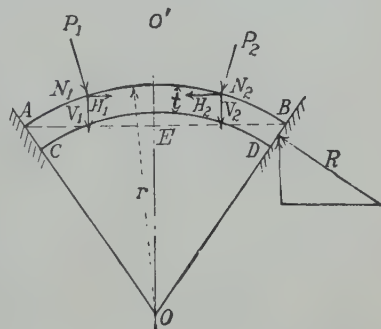


FIG. 118.—Forces acting on arch dam.

Actually, this limit in economical use of the arched dam, in respect to masonry required, will be for a radius somewhat less than 570 ft., due to the greater length of the arc upon which the arched dam must be built, as compared with the chord for the gravity dam. For the usual central angle, say in the vicinity of 120 deg., this limiting radius would be about 500 ft., *i.e.*, beyond which it would be necessary to use practically a gravity section. Note, also, that a smaller allowable value of f , which for high arch dams may be limited to perhaps 25,000 lb. per square foot, will also tend to lower the value of this limiting radius r_c . Such a dam would have a greater factor of safety, however, than a straight dam of the same cross section, as there would be some arch action with the curved dam and evidently the dam could not slide. There are numerous instances, therefore, where a masonry dam of gravity section has been curved in plan, both to increase its stability and to render it more pleasing in appearance. In some cases, too, a curved dam will fit the ledge foundation to better advantage.

Constant Central Angle.—To give a minimum of materials, Jorgensen¹ has shown that a central angle of 133 deg. 34 min. between abutments is required. Practically, owing mostly to cost of forms for concrete, this best angle would normally not exceed about 120 deg.

Using a constant central angle requires a decrease in radius of curvature toward the bottom of the dam and in some cases in a narrow canyon might result in a smaller thickness near the bottom of the dam than at higher elevations, which would not be practicable. This best central angle, however, should always be kept in mind as a guide toward economy and modified only as necessary to meet special conditions.

Arrangement of Spillway.—Obviously, the section of an arch dam would not be suitable for overfalling water; and, if the dam is of considerable height, the only practicable arrangement is likely to be locating the spillway at one side of the dam, or better still, if possible, in an adjacent section of valley somewhat removed from the main structure.

If the depth of water to be handled is not great and, hence, the parabola of overfall rather steep, a spillway section may be designed without much increase in section over that for the abutment section of the arch dam by building an upstream overhang or lip as shown in Fig. 119, which was the arrangement used in the case of the Halligan dam.² This permits curving the downstream face to fit the path of overfall.

The tunnel type of spillway, as used at the Davis Bridge development of the New England Power Company, completed in 1924³ (see also Fig. 289, Chap. XIII) is well adapted for use with the arch dam, where conditions favoring the use of this type of spillway are favorable. These conditions are:

¹ *Trans. A.S.C.E.*, vol. 78, p. 685.

² *Trans. A.S.C.E.*, vol. 75, p. 112, 1912.

³ *Proc. A.S.C.E.*, December, 1923, p. 1953 *et seq.*

1. Sufficient head to permit the use of a tunnel section of reasonable size and cost.
2. Not too great a drainage area.
3. Large reservoir area, to decrease the necessary peak-flood flow.

Present Status of Arch-dam Design.—Of some 40 arched dams for which data are available, nearly all have apparently been designed on the basis of Eq. (9), page 309; *i.e.*, assuming arch action without reference to restraint at the base and with maximum stresses by this formula not usually exceeding 60,000 lb. per square foot even for relatively low dams.

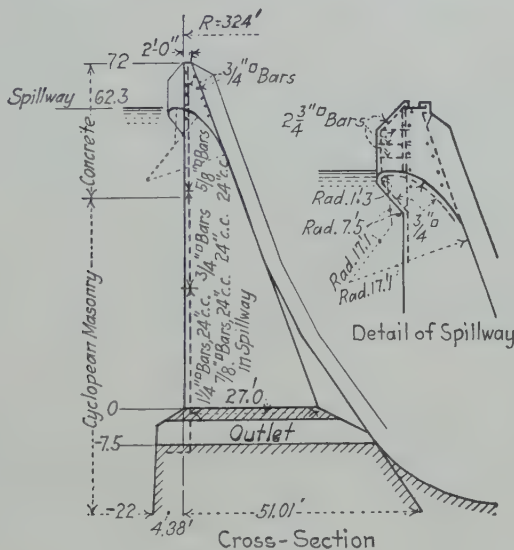


FIG. 119.—Section of Halligan dam.

Some notable exceptions are the Bear Valley, California,¹ Upper Otay, California,² and the Mormon Flat³ dams which have dimensions, stresses, etc., as shown in Table 65.

The Bear Valley dam, as will be noted, is of extremely slender section, but it has withstood a water level nearly at its top. It was replaced in 1910–1911 by a multiple-arch concrete dam built a short distance downstream. The Upper Otay dam relies somewhat upon steel reinforcement, as noted. The Mormon Flat dam, of the Salt River irrigation project, was completed in 1925.

There is no record of failure, as yet, of an arched masonry dam, and Eq. (9) is undoubtedly safe to use, with reasonable unit stresses, say, from 50,000 lb. per square foot for low dams, down to 25,000 lb. per square foot

¹ SCHUYLER, "Reservoirs," p. 163 *et seq.*, 1910.

² WEGMAN, "Masonry Dams," p. 220, 1911.

³ *Eng. News-Record*, May 13, 1926, p. 178.

TABLE 65.—DIMENSIONS, STRESSES, ETC., FOR CERTAIN ARCH DAMS

Item	Bear Valley dam	Upper Otag dam	Mormon Flat dam
Height, feet.....	(1) 48, (2) 64	84	210
Top width, feet.....	2.5 to 3	4	8
Width at base	(1) 8.5, (2) 20*	14	22
Radius of curvature, feet.....	335	359	181
Material.....	Stone masonry	Concrete with steel-plate and cable reinforcement	Concrete
Computed stress, pounds per square foot.....	(1) 118,000 (2) 67,000	134,000	108,000

* At 48 ft. below top, dam increases abruptly by a step from 8.5 to 13.0 ft. in thickness.

for heights exceeding about 100 ft. There has been much discussion, however, of more exact methods of analysis and design of arch dams. Noetzli¹ has developed a method to determine the proportion of the load taken by the vertical cantilever or beam action and by arch action, which, briefly, consists in

... distributing by trial the water (or ice) load on both cantilever and arch system in such a manner that the deflections of identical points, calculated separately for cantilever and arch system under partial load, and with due consideration of shrinkage and temperature deformations, are identical.

He concludes that:

1. His method leads to a fairly close approximation of stresses, at least for the lower portions of a dam.

2. It is proved that certain arches near the crest of high dams evidently are subject to more load than full water pressure alone.

3. The theoretical amount of load carried by the lower horizontal arches, under the temperature conditions assumed, is much smaller, and the stresses in the highest arches are much larger, than present practice assumes.

4. The load on the cantilever, even under consideration of the extra support near the "free" end, is too large to be supported elastically, particularly if the temperature should drop more than 15° F. The cantilever will fail in gravity action, therefore, and produce cracks, either between the dam and foundation or in the masonry itself.

A Committee of Engineering Foundation—a joint organization of the national engineering societies for purposes of research built and tested in 1926 an experimental concrete arch dam about 60 miles east of Fresno,

¹ *Trans. A.S.C.E.*, 1921, pp. 1-135.

Calif., on Stevenson Creek, a tributary of San Joaquin River.¹ This dam had a vertical upstream face with a constant radius of 100 ft. It was built to a height of 60 ft., with the following thicknesses:

Depth below Top, Feet	Thickness, Feet
0	2.0
30	2.0
40	2.58
50	4.40
60	7.50

Its profile along the upstream face was symmetrical, of V-shape with a slight rounding at the bottom.

The test results showed a reasonable agreement between observed and computed deflections. The "cylinder formula" [Eq. (9), page 309] was found to be inadequate for representing the conditions existing in an elastic arch of this type.

Later model tests² of this dam on a scale of 1:12 showed remarkable similarity in deflections calculated from the model to those previously observed upon the full-sized structure and confirmed the practicability of using model tests for such purposes rather than an expensive full-sized structure.

MULTIPLE-ARCH DAMS

The multiple-arch dam is a comparatively recent modification of the hollow dam, usually curved in general plan but made up of a series of arches each springing between buttresses. It is essentially a hollow dam where the flat slab has been replaced by an arch but with a span between buttresses two to three times that for the Ambursen type, or even greater.

With the flat slab or hollow dam, if a slab should fail between any two buttresses, the dam as a whole would not be affected, while the failure of an arch in a panel of the multiple-arch dam might endanger the whole structure. This defect in the latter may be overcome by bracing the buttresses together independently of the arch rings, as was done by Jorgensen³ on certain dams of this type built in California. He found an economical spacing of buttresses arranged in this way to vary from 30 ft. for low dams to 50 ft. for high dams.

It should be noted that a section of arch ring in the multiple-arch dam is not loaded uniformly, due to the inclination of the general upstream face, in the vicinity of 45 deg.

¹ Report on Arch-dam Investigation, *Proc. A.S.C.E.*, May, 1928, and September, 1931.

² "Checking Arch-dam Designs with Models," *Civil Engineering, A.S.C.E.*, May, 1931.

³ *Trans. A.S.C.E.*, March, 1917.

A recent modification of the buttresses, proposed by Noetzli,¹ contemplates making them of hollow, double-wall construction, stiffened sufficiently to avoid buckling, in place of thin solid buttresses with continuous struts between. Such double-wall buttresses may be used to advantage for a dam of ordinary height and to even greater advantage for high dams.

The multiple-arch dam promises to be economical in many locations where rock foundation is available and is attracting favorable attention.

A recent example of a multiple-arch dam on a sand and boulder foundation, but with ledge rock at each end, is the Sherman Island dam on the Hudson River, New York,² already referred to on page 287. It is about 725 ft. long and 81 ft. high, with 31 bays, each 19 ft. center to center of buttresses. The buttresses, $3\frac{1}{2}$ ft. thick, stand on a concrete base 108 ft. wide by 3 ft. thick, extending between the ledge rock at the ends of the dam. The lower portion of the upstream face is inclined about 22 deg. to the horizontal, and the upper 24 ft. 45 deg. to the horizontal. Three concrete ribs under the base, at toe, heel, and near the middle, prevent a tendency to scour and increase resistance for sliding. Two rows of steel-sheet piling are driven as a cut-off at the toe. The average load on the base is about 2.3 tons per square foot, with resultant near the center of the dam. A solid spillway section 864 ft. long, of the usual type, curved in plan and founded on ledge rock, is built on the right bank, with head works for the power canal on the left bank of the river.

ESTIMATES—MASONRY DAMS

For preliminary estimates of masonry dams formulas (7) and (8) (page 303) will be found useful and fairly accurate. These formulas are repeated below with similar formulas applicable to other types of masonry dams.

(Note that Y is cubic yards per linear foot.)

$$\text{Solid concrete dams, abutment sections} \dots\dots\dots Y = \frac{h^2}{66}$$

$$\text{Solid concrete dams, spillway sections} \dots\dots\dots Y = \frac{h^2 + 2Hh}{66}$$

$$\text{Hollow concrete dams, abutment sections} \dots\dots\dots Y = \frac{h^2}{180}$$

$$\text{Hollow concrete dams, spillway sections} \dots\dots\dots Y = \frac{h^2 + 2Hh}{180}$$

$$\text{Plain arched dams, abutment sections} \dots\dots\dots Y = \frac{h^2}{230}$$

¹ *Trans. A.S.C.E.*, vol. 87, p. 342, 1924.

² *Trans. A.S.C.E.*, vol. 88, pp. 1267-1273, 1925.

EARTH DAMS

The earth-fill or earth dam is a general type of structure in very common use. Properly designed and built, it is safe, permanent, and often economical and perhaps resembles, more than any other type of dam, a natural structure, blending into the side hills and forming a harmonious part of the general landscape.

Earth dams are in use with heights varying from small amounts up to the 245-ft. "Cobble Mountain" dam used for the water supply of Springfield, Mass., and incidentally for power development, completed in 1932. Earth-wing dams, usually in connection with masonry dams, have also been used in many developments.

Requisites for Stability.—The cross section of an earth dam is naturally a trapezoid (the form of an earth bank), with a top width varying, perhaps, from 5 or 6 ft. for a low dam to 30 ft. or more for a high dam, and with side slopes depending principally upon the character and stability of the earth fill.

Evidently, under the action of water pressure, an earth dam would not tip over as a whole, although it might slide as a whole if of insufficient weight. If made of ample section to take care of seepage through or under the dam, however, it will be safe against sliding. The dimensions of an earth dam, therefore, are based principally upon practical considerations rather than theory, and upon experience gained by the construction and use of structures of this type.

The essentials in the design and construction of an earth dam are:

1. The foundation must be practically impervious, *i.e.*, of such material and formation that seepage under the dam will be slight, or at least there must be no tendency for water to carry along fine material and, hence, undermine the structure or cause "piping," as it is often called.

2. The dam itself above the foundations must be practically impervious in the same sense as the foundations. This will mean an hydraulic gradient like *AB* (Fig. 120) rather than *AC*. If the gradient were like *AC*, there would be a tendency for the slope to wash at *C* and possibly of fine material being carried along, endangering the safety of the dam by "piping."

3. The side slopes of the dam must be stable under all conditions which may occur during or after construction and evidently must be less than the angle of repose of the material composing the dam. The allowable slopes will vary with the material used for the fill. On the downstream side, slopes may vary from perhaps 1 on 1 for low dams of very stable material, to 1 on 3 or flatter for high dams of ordinary material. Many of the higher dams have been built with a variable slope, starting in at the top, for illustration, with 1 on 2 and changing then

to 1 on $2\frac{1}{2}$ at some distance below the top; then to 1 on 3; then to 1 on $3\frac{1}{2}$, etc., still farther down; with berms or level sections 4 or 5 ft. or more in width at points of change in slope, to help in taking care of surface drainage by gutters and drains and thus avoiding wash of the banks.

The upstream slopes, which will be directly subjected to water action, must usually be somewhat flatter than the downstream slopes at corresponding elevations, because the effect of saturation tends to decrease the angle of repose of the material. A common relation of the two slopes is, to illustrate numerically, 1 on 3 upstream with 1 on $2\frac{1}{2}$ downstream, or a difference of about 1 on $\frac{1}{2}$ between the slopes.

4. Overtopping must never occur; in other words, there must be adequate spillway or gate capacity to handle flood discharge under all conditions. A masonry dam could usually stand overtopping by water without serious damage, but overtopping of an earth dam, due to immedi-

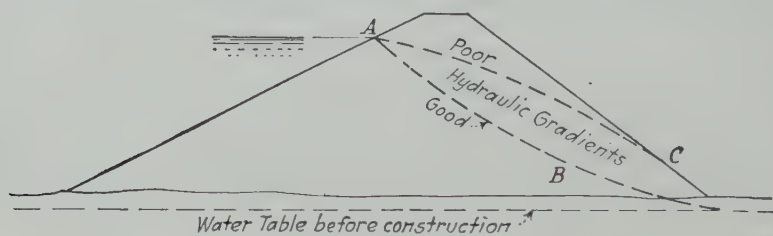


FIG. 120.—Hydraulic gradients—earth dam.

ate washing of channels and carrying away of earth from the top and slopes of the dam, is almost certain to result in its failure. Insufficient spillway capacity is the most common cause of failure of earth dams, as will be noted later, and some of the dam failures which have caused greatest loss of life—notably that of the Johnstown, Pa., dam—have been due to this cause.

The methods of determining probable flood run-off have been fully described in Chap. II, and it is essential here to emphasize only the need of fully providing spillway capacity for a structure of this kind.

5. The outlet pipes through the earth dam, if there be any, must be so designed and constructed that there can be no opportunity for leakage or rupture in a way that would endanger the safety of the dam by washing away of its materials. Next to inadequate spillway capacity, failures due to seepage along, or on account of, outlet pipes have been the most common. Proper arrangements for outlet pipes are further described on page 325.

6. Settlement of the dam after it is put in use should not occur to any great extent. This will usually be slight, if an impervious cross section with stable slopes has been obtained, and is not a common source of trouble except as a result of careless construction.

Sloughing out of a portion of the dam—sometimes during construction or perhaps as the reservoir back of the dam is filled, due to the liquidity or instability of the filled material—is an occasional cause of partial failure, particularly of hydraulic-fill dams. This is usually due to an excess of clay or slippery material and should be guarded against in selecting fill.

Failures of Earth Dams.—The relative importance of certain of the foregoing requirements for stability is clearly shown by a consideration of the cause of failure, where such information is available. Of some 80 failures, mostly in this country,¹ the causes may be classified as follows:

TABLE 66.—CAUSES OF FAILURE OF EARTH DAMS

Cause of failure	Number of failures	Per cent of total number
Insufficient spillway.....	31	40
Outlet pipes, arrangement.....	14	17
"Piping" through dam or foundation.....	14	17
Miscellaneous.....	21	26
Total.....	80	100

Most of these dams were built either without a core or with a core of puddled material; a few had concrete or masonry cores. Of the latter, it is significant that in most cases failure was due to insufficient spillway capacity.

Types of Earth Dams.—Earth dams may be generally classified as (1) with or (2) without a core wall (usually central or nearly so) of impervious material.

Earth Dams with Core Wall. **Concrete Core Walls.**—The most common design is to use a core wall of concrete, usually plain but sometimes reinforced. A typical section of an earth dam with concrete core wall is shown in Fig. 121, with suggested ordinary dimensions or range in these dimensions.

The top width may be greater than that shown where the dam is used as a roadway. The distance above spillway level of high-water level will of course vary with circumstances. The essential is a freeboard or distance between assumed high-water level and top of dam sufficient to allow for wave action and possible uncertainty in the estimated high-water level. For low and unimportant dams, the freeboard may often be less than that suggested.

For dams of ordinary height, say up to 75 ft., materials would usually be hauled in by car or teams, spread in layers, and each layer rolled. It is important to make the upstream half of relatively impervious material,

¹ JUSTIN, "Earth Dam Projects," Table 1.

while the downstream half of the dam should be of coarser or more pervious material, the purpose being to provide drainage for any seepage which may pass the core wall and to keep the hydraulic gradient well below the downstream face of the dam.

The core wall is thin and depends upon the adjacent earth fill for its stability. It should extend to impervious material—ledge rock—unless the latter is at too great depth. Its function is to make a water-tight section longitudinally through the dam (usually at the center); if founded on pervious material, it is practically useless. Its faces commonly slope or batter, as shown, to such a depth as will give a width of 6 to 10 ft., depending on the height of the wall, below which depth the sides are vertical. The bedrock at the base of the core wall should be suitably

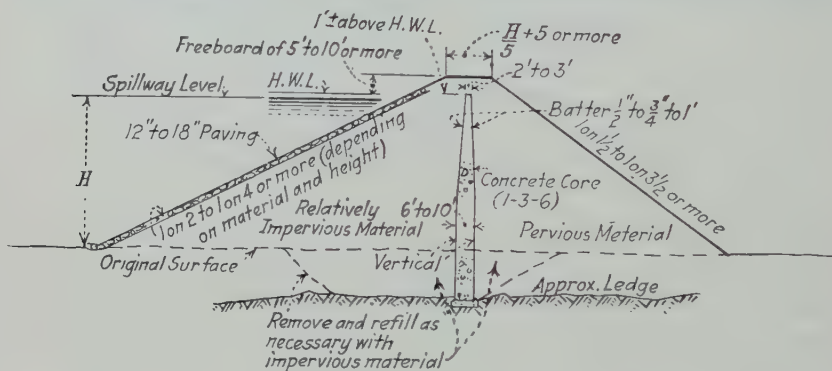


Fig. 121.—Earth fill dam with concrete core. Typical cross section.

prepared—often by excavating a cut-off trench—suitably to bond concrete and rock and prevent seepage at this level.

The Tieton dam¹ upon the Yakima project of the U. S. Reclamation Service in the state of Washington is the highest earth dam with concrete core wall as yet constructed, with an earth embankment 232 ft. high above stream bed. The core wall required excavation to a depth of about 100 ft. to rock foundation and is therefore about 330 ft. high and below ground surface is 5 ft. thick. Above ground surface, it tapers to 1 ft. thick at the top and is heavily reinforced, without expansion joints.

Puddle or Loam Core Walls.—Occasionally, under special circumstances, core walls may be constructed of puddle or clay fill, in which case their top width should be 5 to 10 ft. and upward, with side batters of at least 1 in. per foot.

In a few cases loam or impervious topsoil has been used for the core of an earth dam. The Scituate dam of the Providence, R. I., water supply² is an example of this type of dam, the loam core in this case

¹ *Eng. News-Record*, Dec. 1, 1921, pp. 890–891.

² *Jour. New Eng. Water Works Assoc.*, September, 1922, pp. 323–362.

varying from a width of about 15 ft. at the top to 75 ft. at the original ground level in the maximum section of the dam, which is about 100 ft. high. Below ground level the core is 77 ft. in width, with vertical faces, to ledge rock, a maximum distance of some 60 ft. In this case it was found that a core wall of loam would be more economical than one of concrete. This was due fundamentally to the very pervious character of the main filling material, which would have required loam fill on one or both sides of the masonry core. As the latter would have interfered with such a loam fill, it was concluded to omit the masonry core and depend upon the loam core alone, thus obtaining as tight a dam at a considerably smaller cost than with concrete core.

Where puddle or loam core walls are used, great care must be taken at their base, particularly on ledge foundation, to prevent seepage under the wall. In the case of the Scituate dam above referred to, the ledge was first cleaned, all seams raked out, and loose rock removed (without use of dynamite). A concrete blanket about 30 ft. wide and 1 ft. thick was placed on the ledge, with two concrete cut-off walls 2 ft. square longitudinally, placed near both edges of the blanket. The foundation was also grouted with cement grout, using 20-ft. holes 15 ft. apart. The loam core was then built up in 6-in. layers of rolled loam.

Earth Dams without Core Wall.—Where no concrete or special core wall is used (*i.e.*, a built-up or specially placed core), an earth dam may often be effectively constructed by taking care to place or grade the filled material so as practically to make the inner third or so of the dam of fine, impervious material and the outer portions of relatively pervious material, which, however, will support the central portion.

"Rolled-layer" Method.—In the case of small dams this may be accomplished by care in selection or placing of materials and thorough rolling in relatively thin layers. In larger dams this method becomes too expensive and for them the hydraulic-fill method may be substituted.

It is apparent that the line of demarcation between dams with and without core walls is not sharply defined. In one sense a hydraulic-fill dam may be said to have a core of impervious material. The distinction as made here is in confining the use of the term "core wall" to the (central) impervious portion of the dam, if any part be more impervious than others, whether it be of concrete, puddle, loam, or any other material which is placed separately and not as a part of the process of filling the outer portions of the dam.

Earth dams without core wall, constructed by the rolled-layer process, should in general have the central portion of relatively impervious material, protected by more pervious material in the outer portions of the dam. They should usually be of somewhat larger cross section than that of an earth dam with concrete core. The result should be to minimize percola-

tion and keep the hydraulic gradient well below the surface of the dam at all points, especially near the downstream toe.

Hydraulic-placed or semihydraulic-fill dams are constructed by hauling in material, usually loaded by steam shovel on cars, and dumping it along the line of the slopes of the dam. It is then washed down by a jet of water from a nozzle or monitor, so as to form a central pool of water in which the fine impervious material settles and forms a central section from one-sixth to one-third the width of the dam, backed up on both sides by the coarser and heavier material of the fill.

Control of Materials for Earth Dams.—Sand and gravel, with some rocks and boulders, rock flour, silt, and soils have all been successfully used for hydraulic- and semihydraulic-fill dams. Sufficient fine material—of effective size of 0.005 to 0.02 mm.—is desirable for an impervious core. If too much fine material is present, some should be wasted to avoid slides during construction.

Some laboratory control is essential on earth dams of any importance, to include mechanical and percolation tests of materials, etc. Terzaghi¹ and Gilboy² have developed simple and practical forms of permeameters for the determination of coefficients of permeability by field and laboratory tests.

Somerset Dam.—One of the earlier examples of this type of construction is the Somerset dam, on the Deerfield River, Vermont,³ built in 1911–1913. This dam is about 2100 ft. long and 106 ft. in maximum height, with top width of 20 ft., 1 on 3 upstream slope and 1 on 2.5 downstream slope. The top of the dam has 7 ft. freeboard above maximum high-water level and 11 ft. above full-reservoir level. It contains about 1 mill. cu. yd. of material of excellent composition for such construction, viz., about 20 per cent of clay, 40 per cent sand, 30 per cent gravel, and 10 per cent rock. A concrete outlet conduit 12 ft. wide by 11.5 ft. high was first constructed in ledge rock near the left bank of the river and was used during construction to handle river flow. There was also a riser pipe to take the surplus water from the construction pool. Later, a 48-in. cast-iron pipe was laid in this conduit, with gate house and gate control at the upper end for permanent use. Water was pumped for sluicing under 150 to 175 lb. pressure, and it was found that about 2.8 cu. ft. of water was required for each cubic foot of material. The resulting core occupied approximately the middle third of the dam. Dried samples of this core showed about 94 per cent passing the 200 sieve, and 5½ per cent passing the 100 but not the 200 sieve. This dam has shown little or no leakage during the 11 years it has been in use. Its total cost was about \$800,000. It stores about 2.7 bill. cu. ft. of water at a first cost of about \$300 per million cubic feet.

¹ *Jour. New Eng. Water Works Assoc.*, June, 1929, pp. 191–223.

² *Trans. A.S.C.E.*, 1932, pp. 218–239.

³ *Eng. News*, June 4, 1914, pp. 1236–1241.

The Davis Bridge dam (see Fig. 286, Chap. XIII), about 200 ft. high, which is a recent notable example of this type of dam, as well as several other dams in the East and South, has been built by the same method, which is effective and economical where suitable materials are available.

Records of settlement for the above two dams have been kept since their completion and indicate that this is approaching a condition of rest; especially at the Somerset dam, which covers a period of 15 years.¹

A curve prepared from these records, plotting settlement as a per cent of maximum height against time shows lesser relative settlement as a per cent of height of the Davis Bridge dam on account of greater initial settlement during construction, due to its greater height. The Davis Bridge dam is about 200 ft. high and its core was placed in 8 months, while the Somerset dam of about half this height had its core placed in 16 months. By the date of completion of the former dam the lower 100 ft. had probably received most of its settlement.² The curves appear to be approximately parallel where comparable, and a horizontal with a maximum settlement of a little less than 1 per cent of the height (or about 1 ft.) is apparently being approached by the Somerset curve.

Hydraulic-fill dams are built by both transporting and placing material hydraulically. The monitor and jet are used at the borrow pit, and the material is washed down into flumes or pipes and carried by the flowing water to the slopes of the dam, where it is discharged so as to form a central pool where the fine impervious material collects. Occasionally, sites may be found where water may be impounded in temporary reservoirs at an elevation to permit its use without pumping. More often, however, water must be pumped to obtain the necessary pressure. Suitable material must of course be available at the proper elevation, and, in general, owing to the plant required, this method is economical for dams of considerable size, only those containing about 250,000 cu. yd. or more.

In 1907 Schuyler³ gave descriptions of several hydraulic-fill dams built in the West and also outlined the theory of such structures. According to his suggestions, the inner third of the dam should be impervious, the outer sixths coarse and porous, and the inner half of the outer thirds a mixture of coarse and fine material, to act as a filter and allow slow percolation, but no movement, of fine particles. The best material is gravel, sand, and boulders, with 25 to 35 per cent clay (or, if clay is not available, some other very fine material). The most difficult material to use is clay without sand.

¹ HOLMES, J. ALBERT, "Settlement of Earth Dams," *Eng. News-Record*, Nov. 14, 1929, p. 769.

² EATON, A. C., "Settlement of Dam Cores Occurs during Construction," *Eng. News-Record*, June 25, 1931, pp. 1047-1048.

³ *Trans. A.S.C.E.*, June, 1907, p. 196.

Gilboy¹ in a study of hydraulic-fill dams states that the ideal core should (1) be impermeable, (2) as incompressible as possible, (3) have a high angle of internal friction. These conditions are met in general by graded mixtures ranging from coarse silt to clay, with a high degree of true cohesion, and a small amount of fine sand. A uniform mixture is likely to be unstable under water pressure, and the presence of an appreciable amount of flat-grained particles (as mica) increases compressibility and is entirely unsuitable. Colloidal material should be rigidly excluded and, if construction stops, the central pool should be drained off, otherwise a thin layer of colloidal gel will be deposited, making a plane of weakness in the core.

Some excellent recent examples of hydraulic-fill construction are the five large dams constructed above Dayton, Ohio, by the Miami Conservancy District,² to form detention reservoirs. These vary from 65

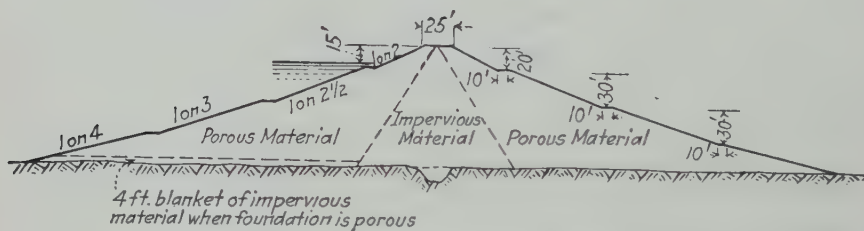


FIG. 122.—Hydraulic fill dams—Miami Conservancy District, typical cross section.

to 110 ft. in maximum height; in length they range from 1200 to 6400 ft. and contain from 790,000 to 3,500,000 cu. yd. of earth, with a total of about 8,000,000 cu. yd. for the five dams, all placed hydraulically. The natural conditions favored the use of earth dams at these sites, ledge rock occurring at one side of the valley only, in each case, while deposits of glacial debris admirably suited for hydraulic sluicing existed in abundance at favorable locations. The cross section adopted for all three dams is shown in Fig. 122, the width of impervious material in the center being arbitrarily assumed at any particular elevation as equal to the distance down from the top of the dam. This in most cases was consistent with the amount of clay and silt available in the borrow material. With one dam, however, proportions were such that some clay had to be wasted from the central pool. Where the foundation material was porous, a 4-ft. blanket of impervious material was placed to increase the ratio of length of travel of seepage to head of water.

A still more recent example of hydraulic-fill construction is shown in Fig. 123, the Toronto reservoir dam near Middletown, N. Y., built during

¹ Thesis, Massachusetts Institute of Technology, 1928.

² *Jour. New Eng. Water Works Assoc.*, December, 1919, pp. 389-435; also *Tech. Reports*, Miami Conservancy District, 1925, Part X.

1925. This view taken during construction shows quite clearly the various essential features of hydraulic-fill work and should be carefully studied. This dam contained about 340,000 cu. yd. of fill, and the pumping equipment consisted of four 1500-g.p.m., motor-operated pumps under 390-ft. head. An 18-in. spiral riveted-steel pipe was used from pump to borrow pits, and a 20-in. high-carbon, welded-steel pipe from pits to dam.

The dam is nearly 100 ft. in maximum height, is 25 ft. wide on top, with 1-on-3½ upstream and 1-on-2½ downstream slopes and has an 8- by 8-ft. concrete outlet conduit, with 50-ft. spillway and waste channel at one end.

The highest dam built as yet by the hydraulic-fill method is the Cobble Mountain dam, completed in 1932, for the Springfield, Mass., water supply and also used for power development. It is 245 ft. high but only 700 ft. long and contains about 1.8 mill. cu. yd. of fill.¹

Use of Core Walls.—Where a core wall can be readily built into an impervious foundation, which may be rock or impervious earth such as hardpan, it is of advantage in minimizing leakage and adding to the safety of the dam. Without a core wall under such conditions but with the impervious layer at or near the surface, after the dam is built and the water level raised, the hydraulic gradient underneath the dam at the toe may be raised to a dangerous extent, *i.e.*, so as to cause a tendency to outflow at this point. This may perhaps be handled by a suitable rock fill at the toe, but the use of a concrete core wall under such conditions is usually desirable. On the other hand, a concrete core founded on pervious material is of little or no use, and the graded-fill type of dam without core is better adapted for such a site.

Arrangement of Spillway.—A spillway of concrete must be used, of course, with an earth dam, and it should be placed to best advantage. This may mean a location in the bed of the stream, at one side, or even in a different valley or hollow in some cases, under which condition the earth dam may be continuous. The presence and location of rock foundation will largely determine where the spillway should be placed. Where the earth dam butts against the spillway section, there must be concrete retaining walls, which will be expensive if very high and will require some care in location.

If the spillway section is located at one side of the river, it will be necessary to provide a channel below the dam to conduct overflow back to the river. This may require some excavation, if in rock; or heavy paving, if in earth.

The tunnel type of spillway, already described (see page 310) as used for the Davis Bridge dam, promises also to be of service for the earth dam under conditions which make its use practicable.

¹ *Eng. News-Record*, July 23, 1928, pp. 124-126.



FIG. 123.—Toronto reservoir-hydraulic fill dam, during construction. (Courtesy of Chas. T. Main, Inc.)

Outlet Pipes and Gate House.—Earth dams built as primary structures, *i.e.*, not as adjuncts or wings of masonry dams, usually require outlet pipes or conduits. Where the pond above the dam is used as a reservoir only, the outlet pipe or conduit would usually be placed at or near the lowest elevation of the river bed back of the dam, to permit complete emptying of the reservoir. Where the development is also for power, the outlet pipe (penstock, in this case) will be located with reference to the lowest proposed draft of reservoir, which is not usually the bottom level.

In either case the outlet pipe or conduit, if possible, should be located in rock (or at least in impervious material) underneath the dam, often at one side of the river. As previously noted, defective arrangement of outlet pipes is a fruitful cause of failure of earth dams and their proper detailed design is important.

The simplest (and poorest) arrangement is a pipe with gate control at the downstream side of the dam, and with several cut-off walls around the pipe in the upstream half to prevent the tendency of water to follow along the pipe. This arrangement provides no means of inspection or repair of the outlet pipe, without drawing down the reservoir; and, if the pipe should break, the safety of the dam is likely to be endangered. Gate control at the upstream end of the pipe, with gate house, will somewhat improve this arrangement, but it is not desirable except in low and unimportant earth dams.

A better arrangement is a concrete conduit (laid in impervious material under the dam and with cut-off walls as necessary) large enough to permit laying a pipe inside the conduit in such a manner that it can readily be inspected and repaired if necessary. Gate control should be at or near the upstream slope of the dam. A steel sluice gate, with provision for stop planks in the gate house above this gate so that it may be inspected or repaired if necessary, is good practice in storage reservoirs where water is to be used for power at other sites downstream. Where power is to be developed at the reservoir site, there must also be provision for trash racks in the gate house. A recent design for such conditions is used at the Davis Bridge development (Fig. 220, Chap. IX), where two hydraulically operated sluice gates are used for control of a 13- by 14-ft. tunnel penstock, with racks permitting a range in water level of about 100 ft.

Relative Cost of Earth and Masonry Dams.—Comparing the quantity of earth fill required for a conventional section of dam, with water depth 50 ft., with that for a masonry dam of the same height it will be found that the yardages of earth and of concrete are in the ratio of about 10:1. Ordinarily, earth fill will not cost more than one-twelfth that of concrete per cubic yard, and less under especially favorable conditions, so that an

earth dam without a core wall is likely to cost less than a solid concrete dam.

With a concrete core wall, however—a fairer basis for comparison, from the point of view of good construction—masonry will be required in amount usually from one-fourth to one-third that for the solid dam, depending on the height. Hence, taking this into account, it will be seen that there will be no great difference in cost between an earth and a concrete dam. More often, it is not so much a matter of cost as of selecting the better type of construction to accord with foundation and other conditions.

ROCK-FILL DAMS

A rock-fill dam is an embankment of loose rock dumped or allowed to fall by blasting approximately into place, with a water-tight section—often an apron of concrete or wood—usually at the upstream slope and properly sealed into the foundation. Such a dam must have a hard foundation which will not be washed out by seepage, and the impervious face or apron must be constructed so as to hold together when the fill settles. A rock-fill dam must also have adequate spillway capacity, as overtopping to any considerable extent would be likely to cause failure.

Dix River Dam.—In 1925 the highest rock-filled dam to date was constructed, known as the Dix River dam of the Kentucky Hydroelectric Company.¹ This is 275 ft. high, of limestone on limestone foundation. Details of the cross section of this dam are shown in Fig. 124. The face wall is of dry rubble, built of large stones, 3 to 8 tons, thoroughly wedged with spalls to avoid abrupt settlements. The concrete facing is designed for flexibility rather than strength. Thus, it is stated that a general deflection of the entire face of the dam, considered as a beam uniformly loaded, to the maximum amount of 10 ft. near the upstream heel where the depth of fill is small and increasing to a deflection of 102 ft. at the crest where the depth of fill is about 275 ft., could safely occur without exceeding safe unit stresses in the slab. Local settlement in individual panels between expansion joints spaced 48 ft. apart could occur to the extent of 1.73 in. near the bottom, increasing to 3.5 in. near the top, for normal working stresses in concrete and steel and about three times this amount of settlement before failure of either. These deflections are all larger than those that have occurred at the Swift dam, near Velie, Mont., which have been from 0.29 to 0.06 ft. in 48-ft. horizontal lengths and a total deflection of 2.7 ft. in a length of 475 ft. It is believed, therefore, that the flexibility of the concrete facing will be ample to accommodate any prospective settlement without resultant cracking and leakage of the facing slab.

¹ HARZA, L. F., "Dix River Dam," *Trans. A.S.C.E.*, 1926; also *Eng. News-Record*, Apr. 2, 1925, pp. 548-552.

very simple to build—no cofferdam is required—the cribwork and sluiceway being built with little or no obstruction to flow. The cribwork is then filled with stone, causing the water to rise somewhat and flow partly through the sluiceway, which is at the lowest part of the stream. Planking is then placed, working both ways from the sluiceway, and filling made of earth seals the planking, as shown. By the time the shallow parts of the stream are reached, all water is being carried through

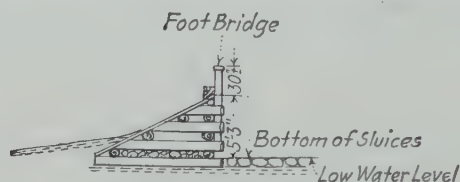


FIG. 125.—Low timber crib dam.

the sluiceway. With the planking completed, the sluice gates are shut, and water is allowed to rise and flow over the crest. It will be noted that this type of dam relies on the weight of stone fill as well as the vertical component of water pressure to hold it in place. Its general cross section is similar to that of the hollow concrete dam.

A somewhat higher timber-crib dam is shown in its spillway section in Fig. 126, the Sewall Falls dam on the Merrimack River near Concord,

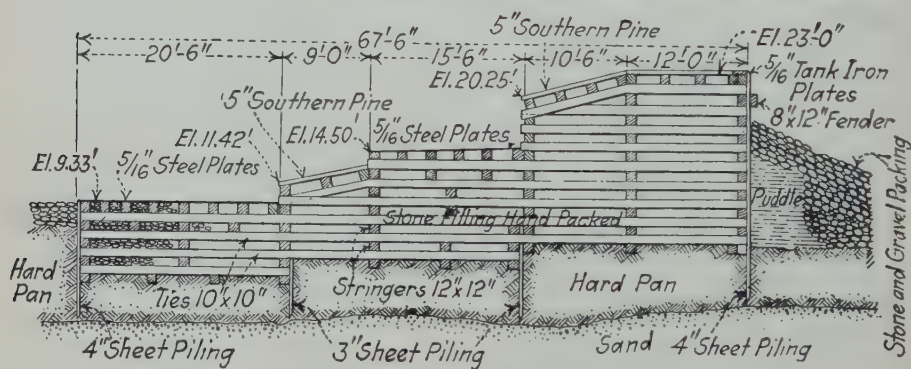


FIG. 126.—Timber crib dam at Sewall Falls, near Concord, N. H.

N. H.,¹ a dam built on earth foundation and still in service. This dam was built in sections, using cofferdams and leaving sluiceways and gates in the completed sections for a water way while finishing the dam. The longitudinal timbers are 12 by 12 in. and the cross-ties 10 by 10 in., mostly hemlock fastened with drift bolts. Spaces were hand packed with stone. Note, also, the steel apron used because of heavy ice. The sheet-piling cut-offs, especially at the heel of the dam, and also a puddle wall protected by stone filling are also important details of this dam.

¹ *Eng. News*, vol. 31, p. 326, 1894.

Timber-crib dams are well adapted for use on earth or relatively soft foundations, as settlement is not harmful and they are readily repaired or leveled up. They cannot be built to heights of more than 30 to 40 ft. owing to heavy pressure developed on the timber and consequent tendency to crush.

Wooden-frame and deck dams have been frequently used for moderate and low heights and are adapted for use on either earth or rock foundation. Figure 127 shows a common arrangement of this type of dam, which resembles the hollow concrete dam in many respects, the support of the deck being by timber bents rather than concrete buttresses, however, and chief reliance being placed upon the water pressure to hold the timber dam in place, whereas the hollow concrete dam has considerable weight to aid its stability.

Owing to the bearing stress developed in the timbers and connections, a wooden-frame dam cannot be used for a dam of considerable height. The cost of lumber, moreover, is now such (taking also into consideration rapid deterioration where the dam is not continuously wet) that, in general, this type of dam is seldom built.

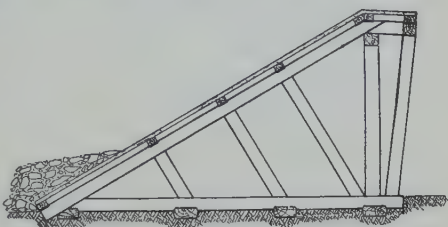


FIG. 127.—Wooden frame and deck dam.

DAMS ON EARTH FOUNDATIONS

Earth dams may often be safely and economically constructed on earth foundations, especially where a relatively impervious material is found at or near the ground surface. Even where there is a more pervious foundation, by widening the dam, trenching the foundation, and filling this trench with impervious material, it is often possible to prevent leakage under the dam as far as is necessary for safety. The principle involved is to make the percolating water travel a long and devious passage, thus keeping the hydraulic gradient down near to its ordinary level in the ground at the toe of the dam.

Hollow concrete dams, as well as wooden dams, may also be built on earth foundations, although their stability in respect to sliding would be less than for rock foundations unless their weight is increased in some way. The concrete mat (with suitable weep holes to relieve upward water pressure) helps to give weight to the hollow dam when built on an earth foundation, while the timber dam can be weighted with stone as far as necessary and provided with cut-offs of sheeting, as was done in the case of the Sewall Falls dam (Fig. 126).

The solid concrete dam is not well adapted for use on anything but a rock foundation, although it may be possible to build such a dam on earth at relatively high cost.

Koenig¹ stated the essentials for a masonry dam on a sand foundation, as follows:

1. Water must not flow through the sand foundation so as to wash it away.
2. The stream bed at the downstream side of the dam must be protected to avoid wash and undermining of the toe. Sand will carry 5 to 8 tons per square foot when confined but washes very readily.
3. The section must be ample to withstand sliding (keeping in mind the considerable upward water pressure that may exist in such a case).

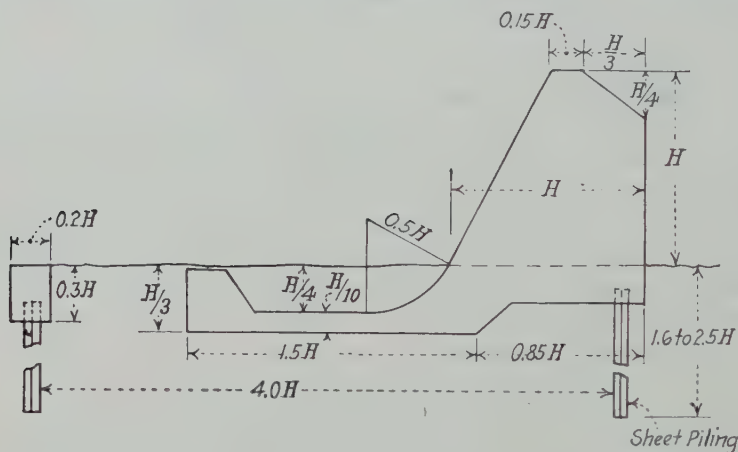


FIG. 128.—Section of masonry dam on sand foundation. (Koenig.)

He suggests a spillway section for a concrete dam as shown in Fig. 128. The features of importance are the cut-off of sheet piling at the heel; the ample cross section, base width equal to the height; the pool at the toe of the dam; and the farther wall and sheeting downstream from the toe.

Such a structure is obviously more hazardous than when built on rock foundations and is also very expensive; hence, locations requiring such construction should always be avoided if possible.

ROCK FOUNDATIONS OF MASONRY DAMS

A hard igneous or metamorphic rock, such as a granite or a syenite, makes the best foundation for masonry dams. Limestone is likely to have fissures and to be more or less unreliable. A sandstone or mica schist, being stratified, is always seamy and difficult to make tight. The strata may also be tilted in such a manner (downstream) as to give

¹ *Trans. A.S.C.E.*, January, 1911, p. 175.

the dam less stability against sliding, or the seams may run approximately at right angles to the axis of the dam thus tending toward leakage under it.

In preparing the foundation, loose or weathered rock should be removed and the sound rock surface thoroughly cleaned. Usually, a cut-off trench should be excavated near the heel of the dam, varying from a trench 3 or 4 ft. square to one 10 or 15 ft. or more deep and of corresponding width in the case of a high dam.

Grouting Foundations.—In the case of seamy or poor rock, it may be necessary to grout a cut-off section of the foundations near the heel of the dam, sometimes in the cut-off trench itself.

The process of grouting consists of drilling holes in the rock with a core drill and, by compressed air, forcing cement grout (from 2 to 4 parts of water to 1 of cement, depending upon circumstances) into the hole and its adjacent seams. This is usually done by means of the Canniff grouting machine, which is an arrangement of two grout tanks, with pipe connections so that grout from one may be subjected to air pressure from an air compressor, a necessary part of the outfit, and "shot" into the hole, while the other grout tank is being filled with a new charge. Air pressure from 100 to 200 lb. or more per square inch is used.

Garratt¹ describes in detail the method of grouting the foundations of the Nepaug and other dams of the city of Hartford water supply and suggests as a result of their experience the following procedure:

1. Drill two rows of primary holes 7 ft. apart with holes 15 ft. apart in each row and staggered, the upstream one of these rows to be located just under the upstream face of the dam. Test and grout these holes.
2. Drill, test, and grout secondary holes as needed between these.
3. Drill, test, and grout tertiary and even quaternary rows of holes if needed finally to prove the grouting.

4. After this grouting is finished, section by section, if the work is so carried on, drill vent holes about 15 ft. back of the upstream face of the dam, spaced 10 ft. apart, of such depth as the testing of the grout holes shows to be necessary, and carry these by pipe up into the inspection gallery of the dam to be observed, tested, and grouted or treated as needed after the water is ponded against the dam.

5. If there is to be considerable head of water on the rock foundation due to the water standing on the downstream side of the dam, in so far as is reasonably possible, by the removal of loose rock, by drilling and grouting of holes, or by placing of vent pipes at outflows to be grouted later, make tight the rest of the rock bottom below the elevation to which water will stand on the downstream side.

6. Above the elevation to which water will stand on the downstream side, unless springs under pressure are encountered, make no attempt to tighten the rock bottom downstream from the upstream grouted section.

¹ *Jour. New Eng. Water Works Assoc.*, December, 1917, pp. 609-618.

CONTRACTION JOINTS, ETC.—MASONRY DAMS

Being monolithic, all concrete structures, if of considerable length, will crack due to restrained shrinkage as temperature lowers. It is desirable, therefore, to construct a dam in comparatively short sections, say from 30 to 50 ft. in length, with so-called "contraction joints" between these sections. This may be done effectively as shown in Fig. 129, the plane of the concrete between successive sections *M* and *N* being broken by triangular notches as shown, formed by timbers fastened to the forms. By the simple device shown, a small vertical notch will be left in the concrete at *C* and *C'*, so that the crack forming at this point will be inconspicuous. Before concrete is poured in section *N*, the

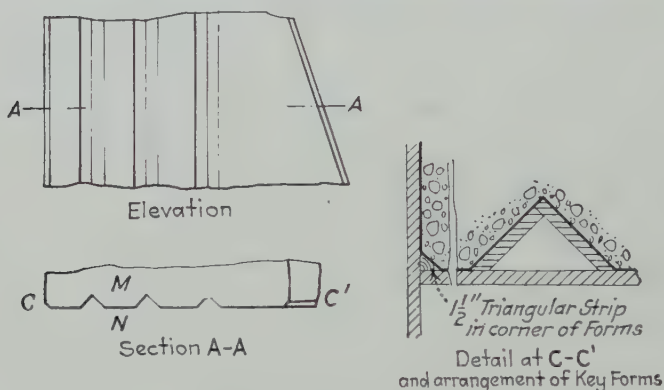


FIG. 129.—Contraction joints—masonry dams—details.

face *CC'* should be covered with an asphalt paint, to insure the crack forming at this vertical plane. If concrete is poured in cool weather, there will be little or no seepage through such a joint after water is allowed to rise in the dam. In any event, such joints will soon tighten with silt or cement "laitance," and seepage cease.

Horizontal joints, as between successive days' pouring of concrete, should be keyed by placing good-sized rocks so as to be embedded in the concrete first poured, which will project and later be surrounded by the next section of concrete.

Care must also be taken always to clean the surface of old concrete thoroughly and give it a wash of cement grout before placing new concrete.

Finish and Appearance of Dams.—While ornamentation of a masonry dam is not usually desirable, except occasionally in municipal or governmental projects, care should be taken in the design of dams for water power developments to obtain, as far as practicable, a structure of dignified appearance appropriate to its surroundings.

Often, the general appearance of a masonry dam may be improved, with little increase in cost, by curving it somewhat in plan. Where the

spillway is at one side of the river, with sloping channel to the river below the dam, the downstream face may also be stepped, to improve its general appearance.

The top of the bulkhead section, if arranged as shown in Fig. 130, will be much improved in appearance over that built strictly rectangular in cross section. Concrete parapets may also be used for the bulkhead sections of masonry dams, particularly in the case of a hollow reinforced-concrete dam.

The use of triangular notches at contraction joints has been suggested previously. The vertical joints should be laid out at approximately regular intervals so as to give the effect of panels upon the faces of the dam.

The exposed slopes of earth dams should always be finished with a layer of loam and well seeded down, not only to obtain the better appearance of a grass slope but to help to prevent gullies forming due to wash from heavy rains. Clearing up after construction is especially important, and unsightly borrow pits in the vicinity should be avoided if possible.

Construction Cost of Dams.—In estimating the cost of a dam it is usually advisable to consider the subdivisions of the structure separately, estimating and tabulating the different classes of items and applying suitable unit costs thereto. The selection of proper unit costs requires the best judgment and should be attempted only by engineers of adequate experience in this field. Obviously, such unit costs will vary through a wide range, depending upon the quantities involved, local conditions, special difficulties, etc., which affect costs.¹

Table 67 gives typical items likely to occur in masonry and earth-dam construction, with an approximate usual range in unit costs, intended to cover all construction costs, including contractor's profit but not the cost of engineering for which an allowance of 15 per cent, to include contingencies, should be made. The figures give approximate costs on about a 200 per cent cost index.² They are intended, however, to give only a general idea of unit costs, and limitations in the use of such data should be kept in mind as noted above.

¹ Part X, *Tech. Reports*, Miami Conservancy District, 1925, contains excellent detailed cost data of hydraulic-fill dams and concrete structures.

² See Table 122, Chap. XII.



FIG. 130.—Top arrangement masonry dam, bulkhead section.

TABLE 67.—ITEMS OF COST AND UNIT COSTS—MASONRY AND EARTH DAMS

Item	Unit	Range in unit price, dollars	Remarks
1. Earth excavation (or borrow).....	cubic yard	0.50-1.00	
2. Earth fill.....	cubic yard	0.75-1.25	
3. Rock excavation.....	cubic yard	4.00-6.00	
4. Concrete masonry.....	cubic yard	20-30	Class A, 1-2-4 reinforced
	cubic yard	15-20	Class B, 1-2-4
	cubic yard	10-15	Class C, 1-3-6
	cubic yard	10-15	Class D, cyclopean
5. Steel reinforcement.....	pound	0.04-0.07	
6. Miscellaneous iron and steel.....	pound	0.08-0.12	As for beams, gratings, railways, racks, flashboard pins, etc.
7. Stone paving.....	cubic yard	3-5	Without mortar, as for slopes of earth dams

Items for which special estimates must always be made are:

8. Clearing and grubbing, usually not a large item unless a reservoir is included.
9. Handling water, may vary from a few hundred dollars upward, depending on size of river.
10. Gates and controls (see page 517, Chap. IX).
11. Gate house, usually 20 to 30 cts. per cubic foot of volume, depending upon type of construction.

References

In this chapter upon dams only the basic principles and some of the essential facts regarding the design and construction of dams have been presented. The student and designer should consult some of the more complete treatises on this subject, among which are:

CREAGER, "Masonry Dams," John Wiley & Sons, Inc., New York.

HANNA and KENNEDY, "Design of Dams," McGraw-Hill Book Company, Inc., New York.

JUSTIN, "Earth Dam Projects," John Wiley & Sons, Inc., New York.

WEGMAN, "Masonry Dams," John Wiley & Sons, Inc., New York.

CHAPTER VI

THE WATERWAY—CANALS AND PENSTOCKS

In the type of water power development with extended fall, where additional head is developed over and above that created by the dam alone, a waterway channel is required to carry water to the wheels at the power house. As has been previously noted, depending upon various factors, this waterway may consist of an open channel or canal, a closed conduit or a pipe line (or penstock, as it is often called), or a combination of the two.

As in the other features of a water power development, the waterway should produce the desired result at a minimum of cost consistent with proper design or safety. Hence, in general, the canal will be used where the water to be used is large in amount, because a pipe would be too large and too costly, although the limitations in respect to topography along the river banks may sometimes be a dominating factor.

For a mean velocity of 8 ft. per second—an ordinary figure for flow in penstocks—the capacity for various large sizes of pipe would be:

Diameter of pipe, feet	Area of pipe, square feet	Discharge, $V = 8$ ft. per second
9	64	512
10	78	625
11	95	760
12	113	920

For a size of pipe over 9 or 10 ft. in diameter, depending upon the length, the cost is likely to exceed that of an open channel or canal, and an ordinary limitation in use of pipe may be placed at about 600 to 800 sec.-ft. In many instances, however, especially in the West, canals, or open ditches, as they are often called, have been used of relatively small capacities, many of but 200 to 300 sec.-ft. capacity and some as low as 50 sec.-ft. A border-line case, therefore, must be carefully studied, in view of any special conditions affecting cost, and decided by itself.

CANALS

Cross Section.—This will vary with the material through which the canal is constructed. In rock the section would be approximately rectangular, while in earth it would be trapezoidal, with side slopes usually

from 1 on 1 to 1 on 3, depending upon the material. The slopes must of course be stable.

For best hydraulic section, whether rectangular or trapezoidal, the hydraulic radius is one-half the water depth. (In the case of the rectangular section, this also means that the width is twice the depth.) The best hydraulic section will not necessarily be the most economical section, although often not differing greatly from it.

The area of cross section and, hence, the velocity of flow for a given discharge may depend on (1) the allowable loss of head or slope of water surface in the canal such that the annual cost of canal plus power lost in friction head is a minimum, *i.e.*, the best section of canal; or (2) some limitation in velocity necessary to avoid scour, or washing of banks, in the case of unlined canals in earth.

1. To determine the best section of canal, the cost per foot (or for the full length of canal) of several different sections of varying depth should be carefully estimated and the annual cost of each section computed based on a suitable percentage allowance covering fixed charges of interest, taxes, depreciation, maintenance, etc., usually totaling from 9 to 11 per cent, depending on the type of canal and other construction conditions.

The loss of head per linear foot (or for full length of canal) should be estimated for the average discharge expected. The latter corresponds to the plant capacity multiplied by its capacity factor, which in turn depends principally on its load factor.

The power loss may then be computed in horse power using the combined efficiency of wheels and generator, or, in other words, as if at the switchboard. The value of this power lost per horse power is the difference between its cost and its sale value at the switchboard. Its cost will include operating costs and fixed charges on the plant but in this case not including the cost of the canal itself. Its sale value may not be definitely known, especially in the case of a new project, but should be estimated as accurately as possible. The value of the power in a water power development is really identical with the value of the water privilege or undeveloped power (not including the canal or taking it into account, however, in this particular case).

The total annual cost for each assumed depth of canal will be the cost of canal plus value of power lost. By plotting a curve of annual cost as ordinates, against depth of canal as abscissas, the minimum point on the curve or depth giving the least annual cost may be readily determined.

While the exact application of this method may be difficult in some cases, due to uncertainties as regards cost estimates and value of power, it is fundamentally sound in principle and of great value not only for the determination of the best section for a canal but of pipe line or penstock as well (to which it is even better adapted). The same principle under-

lies the determination of the best size of wire for transmission lines, and it may also be used in problems relating to pumping water through pipe lines and, in fact, any case where the size of pipe or conductor affects the passage of the medium under transmission.

2. Limitations in velocity, which usually are of importance only with unlined canals in earth, must bear in mind not only average velocity but maximum velocity, or that corresponding to peak load as well. Scouring velocities will vary with the material. Ordinary limits are about as follows:

	Feet per Second
Ordinary velocity allowable.....	2-3
Velocity in sand.....	1
Velocity in hard gravel or clay.....	4-6

Too low a velocity is not desirable in earth-lined canals on account of tendency to deposition of silt and also to the growth of weeds or other aquatic vegetation.

Studies of canal velocities may be based upon the use of either the Chézy-Kutter formula

$$V = C\sqrt{RS} \quad (1)$$

or the Manning formula

$$V = \frac{1.486}{n} R^{2/3} \sqrt{S} \quad (2)$$

where n is practically the same as in the Kutter's formula.

Practical Canal Sections.—In level ground the canal section may be based on the best hydraulic section for the side slopes allowable for the particular material encountered modified in the case of large canals by the fact that in general the cost of excavation will increase somewhat with the depth, this tending slightly to widen the cross section and make its depth somewhat less than that for the best hydraulic section.

In side-hill locations, however, the tendency is in the opposite direction, as will be noted from Fig. 131. For the same water area (or, more strictly, the same hydraulic radius) section 1 will be more economical than section 2. If the areas A are approximately the same in each case, section 2 will have an excess area of excavation B due to the side slopes. Evidently, in side-hill locations the most economical section will be somewhat deeper than the best hydraulic section. This is true either in earth, for the trapezoidal section; or in rock, for the approximately rectangular section.

On steep slopes there should be a berm at a point somewhat above water level, to prevent material from falling or sliding into the canal.

Location.—The "economically shortest route" is the ideal to keep in mind in canal location. This will be some line intermediate between (1) a location following the contour of the side hill from dam to forebay,

with a minimum of excavation or fill, resulting in a very long canal, and (2) a straight line from dam to forebay, which would usually result in excessive cuts and fills, the cost of which would much more than offset the saving due to the straight-line location.

The water-level grade of a canal is restricted to a relatively small range of elevation, as hydraulic slopes are slight in such channels, and the slope, of course, must always be toward the power house and usually substantially uniform in amount. Canal location, therefore, is much restricted, more so than a railroad location, as the latter, while limited to certain grades (which, however, are much steeper than hydraulic grades), may have these running in either direction.

At the dam or intake of the canal, the location must be at such an elevation as to provide for any probable fluctuations in water level, particularly low water. The water surface in the canal near the dam

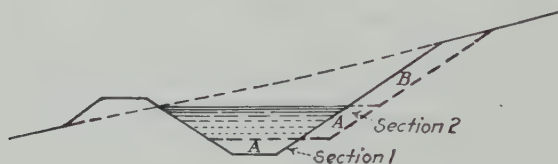


FIG. 131.—Canal sections.

may often be below the crest level of the dam, where pondage is of importance, and usually provision is desirable for a minimum water level in the canal of 2 to 4 ft. and often even more below the crest of dam. Conditions of probable draft of pond must always be carefully considered, each case by itself, in fixing the canal grade. It should be kept in mind, furthermore, that location will be limited by the maximum hydraulic gradient (or that corresponding to peak-load or full plant capacity), whereas, as already noted, the size of cross section is based fundamentally on average load.

A canal must also be arranged to handle high-water conditions safely and economically. High water may be provided for as follows:

1. By making the canal of adequate depth of cross section to provide for the entire range of water level between high and low water. This is not usually economical or desirable, as the resulting section of canal would be too large and expensive.

2. By throttling flow into the canal at the head gates by partially closing the gates. This is effective but requires care and watchfulness in operating.

3. In addition to partly checking the flow by the use of the head gates, a wasteway or canal spillway may be provided to prevent too great a rise of water in the canal. This method is frequently used and may also be supplemented by the use of one or more waste gates, in the canal,

which discharge into the river, commonly placed for convenience in the forebay near the power house.

Where the dam is used for storage of water, as well as for power development, the use of a canal is likely to be undesirable, owing to the great range in water level between full and empty reservoir. If a canal is used, it must be located low enough to take care of flow at low reservoir level, and this would commonly mean throttling the flow at higher levels with the gates and, hence, undesirable loss of head. In such cases a penstock should be used, set at the low level but capable of utilizing the available head at any level.

It is evident that the canal location and its most economical cross section are more or less interdependent and will require considerable study in a project of any importance, with consideration of perhaps several different lines and a comparison of their relative costs, before a final decision can be made.

Lined Canals.—Canal linings may be desirable (1) to reduce loss of head and, hence, size of cross section, and (2) to prevent scour of banks and bottom, particularly at maximum velocities.

1. Linings may be of stone paving or riprap, usually laid dry, of a thickness of 12 to 18 in. Such a lining is chiefly effective in eliminating scour of banks and bottom and adds little or nothing to the capacity of the canal.

2. Concrete linings are not only effective in eliminating scour but also add very materially to capacity. Thus, the ordinary values of n in the Kutter's formula are for unlined and lined canal sections:¹

HORTON'S VALUE OF n IN KUTTER'S FORMULA

Canals and ditches:

Earth, straight and uniform.....	0.0225
Rock cuts, smooth and uniform.....	0.033
Rock cuts, jagged and irregular.....	0.040
Winding, sluggish canals.....	0.025
Dredged-earth channels.....	0.0275
Canals with rough, stony beds, weeds on earth banks.....	0.035
Earth bottom, rubble sides.....	0.030
Concrete-lined channels.....	0.015

Keeping in mind that the value of C in the Kutter formula (and, hence, for a given section the value of V and Q) varies practically inversely as n , through any given reasonable range in hydraulic radius [C varying about as $(R)^{1/6}$], it will be seen that the use of a concrete-lined canal will practically double the capacity, as compared with either an unlined or stone-paved earth section. In other words, if the canal is lined with concrete,

¹ HORTON, R. E., "Some Better Kutter's Formula Coefficients," *Eng. News*, Feb. 24 and May 4, 1916.

subsequently removed and the joints grouted with a cement gun. Most of the canal section is through sand, and, after filling and quickly emptying the canal, weakness was disclosed in the downstream end of the lining, due to ground water and leakage, and this portion was strengthened by building concrete steps on the north (land) slope and concrete struts over the bottom slab.

Effect of Ice on Canal Capacity.—In cold climates it becomes necessary to consider the effect of ice cover in determining the capacity of a canal. The effect is practically to double the wetted perimeter and, hence, for a given area to reduce the hydraulic radius one-half. There is also some increase in friction head due to the greater frictional effect of the ice surface (underneath) as compared with air surface.¹

Neglecting the latter, or in other words, assuming the same value of C for ice cover as for open section, there would result a loss of capacity due to ice cover of about $(1.0 - \sqrt{0.5}) =$ say, 0.3, provided the area of canal and hydraulic slope remain the same.

To maintain discharge under ice cover, keeping the same water area, would require increasing the slope in the ratio of about 2 to 1, which could not be done usually in a canal of any considerable length. The water area might be somewhat increased, but, to offset the ice effect, again the area would have to be practically doubled, which is not usually possible. Obviously, if ice cover is likely to prevail, its effect must be carefully taken into account in designing the canal section.

Where water flows in a regular channel with an appreciable velocity, ice formation always originates in the low-velocity portions of the cross section near the shores. Having secured the shore line for an abutment, this marginal ice builds out, bridge like, from both shores toward the center of the stream, the rate of progress being mainly governed by the climatic factors above mentioned. While these marginal ice sheets build outward, they also thicken, and, if the climatic factors are such as to cause them to meet in the middle of the stream, the under surface of the sheet is roughly parabolic in form, having a minimum thickness at midstream and a maximum thickness at the haunches abutting the shores.

In climates where there are sustained periods of low temperatures, as in many parts of Canada, a complete ice cover may be formed in this way and persist for considerable periods; but, if the temperature rises sufficiently, the ice sheet at midstream disappears through the joint action of temperature and erosion, and open water will show. This thread of open water will widen out, get narrower, or close together, in harmony with the variation in the climatic factors. The haunch ice, meanwhile, holds intact and tends gradually to thicken through the agency of ordinary frost action and also by reason of accretions of slush

¹ U. S. Geol. Survey, *Water Supply Paper* 187, pp. 14-19.

and frazil which adhere to the lower surface of the ice sheet, as a result of the low transportivity of the shoreward current filaments. Hence, tendency to ice formation is at a maximum for wide, shallow canal sections with flat slopes, which should, therefore, usually be avoided.

To reduce ice effect further and particularly the formation of frazil and anchor ice, the water should be kept flowing as regularly as possible, avoiding sudden changes in cross section, interruptions due to piers, etc.¹

Canal Bends.¹—Designs used on the Chippewa project of the Ontario Hydroelectric Commission are consistent with the following theory: (1) That the loss induced by the deflection of the streamlines will be a minimum, if the agency producing the disturbance, namely, the curve itself, is made as short as possible; and (2) that the turbulence induced by the change in direction of flow will be reduced and more or less localized, if extra lateral space is provided at the seat of disturbance, so that whirls and eddies may be quickly dissipated instead of being forced forward into the straight section of the waterway. The first requirement is met simply by making the radius of the curve as short as possible; and the second by making the curve equiradial. The results of construction according to these rules, and actual operation, appear to justify their soundness.

Examples of Power Canals.—In Table 68 details of a number of power canals in various places are given. Data for those in California and Nevada have been compiled from *Water Supply Paper* 493,² U. S. Geological Survey. These canals vary in length from the very short canal at Plant 3 of the New England Power Company, Massachusetts, to the 21½-mile canal of the Halsey plant in California. As previously noted, some of the California canals are of small capacity, and the prevailing water slope of the long canals is from ½ to 1 ft. per thousand. In hard material side slopes as steep as 1 on ½ have been used, but the tendency appears now to be toward the use of a concrete lining. Hydraulic radius is from about 0.5 to 0.8 the water depth, commonly being about 0.6. Velocities in earth sections seldom exceed 3 ft. per second as a maximum, unless the material is hard or rocky. In the case of concrete linings, 4 or 5 ft. per second may be economical.

Accompanying Table 68 are notes for each plant giving further details of waterway, plant capacity, etc.

NOTES ACCOMPANYING TABLE 68

1. Turners Falls, 1916. Total length of canal, head gates to power-house intake, about 2.2 miles; forebay of about 40 acres for about ¾ mile above power-house; wheels, six vertical units, each 11,670 hp., 97.3 r.p.m., 60-ft. head.

2. New England Power Company 3, 1913. Total length waterway, 0.32 mile, consisting of 0.11 mile covered concrete 12.5 by 17 ft., 0.17-mile canal and 0.04 mile of

¹ *Eng. Jour.*, Canada Eng. Inst., July, 1924, pp. 388-391.

² FOWLER, F. H., "Hydroelectric Power Systems of California," 1923.

TABLE 68.—DIMENSIONS OF CANALS—WATER POWER DEVELOPMENTS

Plant	Location	Length, miles	Grade, feet per thous- sand	Water depth, feet	Bot- tom width, feet	Side slopes, 1 on	Material		Hydraulic properties						
							Sides	Bottom	A	P	$\frac{R}{D}$	Maximum		Ordinary	
												Q	V		Q
1 Turners Falls.....	Connecticut River, Massachusetts	$\left\{ \begin{array}{l} 0.63 \pm \\ 0.63 \pm \end{array} \right\}$		30 23	86 123	1½ 0.083	Stone pav- ing Masonry walls	Earth Rock	3,920 2,870	215 169	18.2 17.0	0.60 0.74	12,000 15,000	3.1 5.2	
2 New England Power Com- pany 3.....	Deerfield River, Massachusetts	0.17		19	35	2	Stone pav- ing	Earth	1,380	120	11.5	0.60	1,760	1.27	1,300 ± 0.95 ±
3 New England Power Com- pany 5.....	Deerfield River, Massachusetts	1.9		15 to 10	30	2	Earth	Earth	900 to 500	78 to 75	10.0 6.7	0.67 0.89	970 1.9	1.1	
4 Sherman Island, ultimate.....	Hudson River, New York	0.6		26	32	1½	8-in. con- crete lin- ing	8-in. con- crete lin- ing	1,846	126	14.7	0.57	7,500	4.1	
5 Falls Village.....	Housatonic River, Connecticut	0.38	0.40	10.5	23	1 (R) and 0.3 (L)	6-in. con- crete lin- ing	6-in. con- crete lin- ing	389	54	7.2	0.69	2,500	6.4	
6 Drum, ultimate.....	Bear River, California	8.8		7	11	$\left\{ \begin{array}{l} \frac{1}{2} \\ \frac{1}{2} \end{array} \right\}$	Stone walls	Earth	$\left\{ \begin{array}{l} 10128.6 \\ 12630.8 \\ 8925 \end{array} \right\}$	$\left\{ \begin{array}{l} 3.80 \\ 4.10 \\ 3.60 \end{array} \right\}$	$\left\{ \begin{array}{l} 0.54 \\ 0.58 \\ 0.53 \end{array} \right\}$	600 6.0 4.8		350 4.0	
7 Halsey.....	Dry Creek, California	$\left\{ \begin{array}{l} 5.5 \\ 16.0 \end{array} \right\}$	1.10	6.7	10	½	Partly stone walls	Earth	118	28.4	4.2	0.51	...	...	350 3.0
8 Borel.....	Kern River, California	$\left\{ \begin{array}{l} 8.0 \\ 1.8 \end{array} \right\}$	0.20	7.5	23	1½	Concrete stone walls	Concrete lining	252	49.4	5.1	0.68	...	...	590 2.3
9 Kaweah 2.....	Middle Fork, Kaweah River, California	3.1	1.0	7.5	15	1	Concrete lining	Concrete lining	168	36.2	4.6	0.61	...	...	590 3.5
10 Tule River.....	Tule River, California	2.1	1.0	3.5	5.0	1	Concrete lining	Concrete lining	29.6	15.0	2.0	0.57	...	...	80 2.7
				3	4.5	1	3-in. con- crete lin- ing	3-in. con- crete lin- ing	22.5	13.0	1.7	0.58	...	...	50 2.2
11 Verdi.....	Truckee River, Nevada	2.0	0.4	6.5	17	1	Earth	Earth	152	35.4	4.3	0.66	...	...	500 3.3
12 Washoe.....	Truckee River, Nevada	$\left\{ \begin{array}{l} 1.4 \\ 0.4 \end{array} \right\}$	0.45	6.0 8.0	12 45	1 ½	Earth Rock wall	Earth River side	108 392	29.0 61.6	3.7 6.4	0.62 0.80	...	...	300 1,200
13 Folsom.....	American River, California	$\left\{ \begin{array}{l} 0.6 \\ 0.7 \end{array} \right\}$	1.0 1.0	8.0 8.0	40 40	0.62 0.62	Rock faced Riprap on river side	Rock faced Riprap on river side	360 360	62.4 62.4	5.8 5.8	0.72 0.72	...	...	1,200 1,200
14 Seros.....	River Segre, Spain	11.8 ±	0.15	13.1	18.7	1¼	4-in. con- crete lin- ing	4-in. con- crete lin- ing	460	60.7	7.6	0.58	...	...	2,100 4.56

three 10-ft. penstocks; wheels, three horizontal units, each 3200 hp., 257 r.p.m., 64-ft. head.

3. New England Power Company 5, 1915. Total length of waterway, 3.25 miles, consisting of 1.25 miles of tunnel and concrete penstock 12 ft. in diameter, 1.9 miles of canal (canal and penstock alternate, the latter on steep hill slopes) and 0.1 mile of three 7-ft. penstocks; wheels, three horizontal units, each 7000 hp., 300 r.p.m., 240-ft. head.

4. Sherman Island, 1923. Total length waterway, 0.8 mile, consisting of 0.76-mile canal section (and forebay) and 0.04 mile of 10 by 10½-ft. concrete penstocks; wheels, four vertical units (eventually five), each 10,000 hp., 150 r.p.m., 66-ft. head.

5. Falls Village, 1914. Total length of waterway, 0.45 mile, consisting of 0.38 mile of canal and 0.07 mile of three 9-ft. penstocks; wheels, three horizontal units, each 5000 hp., 300 r.p.m., 90-ft. head; upper 260 ft. of canal has spillway.

6. Drum, 1913. Total length conduit, 8.8 miles, all canal except four flumes, totaling 0.5 mile, and two siphons: (1) 0.36 mile of steel pipe 8½ ft. in diameter, (2) 0.36 mile wood-stave pipe, mostly 8 ft. in diameter; penstock of steel pipe, one 72 in. in diameter, 1.2 miles long (eventually two lines); wheels, two units (eventually four), each 85-in. double-overhung Pelton wheels, 18,000 hp., 360 r.p.m., about 1300-ft. head.

7. Halsey, 1916. Total length conduit, 23.4 miles, with 21.5-mile canal, 0.5-mile timber flume, and 1.2-mile tunnels about 8 by 8 ft.; forebay of 185 acre-ft. capacity; pressure system, 1.0 mile of tunnels and 72-in. steel pipe; wheels, two units, each 60 in. horizontal, 2000 hp., 360 r.p.m., about 300-ft. head.

8. Borel, 1904. Total length conduit, 11.2 miles, with 9.8-mile canal, 1.0 mile of flumes, and 0.4 mile of tunnels; wheels, originally five units Girard type, now one 3600 hp. Doble impulse, and four 4000 hp. horizontal reaction, 231 r.p.m., about 260-ft. head.

9. Kaweah 2, 1905. Total length conduit, 4.0 miles, with 3.1-mile canal and 0.9-mile wooden flume 5 ft. in width by 4 ft. in depth; penstock 0.2 mile 60- to 40-in. steel pipe; wheels, one 32-in. horizontal, 2250 hp., 720 r.p.m., about 360-ft. head, and one 39-in. Victor Girard, 1000 hp., 450 r.p.m.

10. Tule River, 1909. Total length conduit, 6.8 miles, with 2.1-mile canal, 4.5-mile 4- by 3-ft. wooden flume, a short concrete flume and siphon; small forebay and sand trap; penstock 30- to 24-in. steel pipe 0.5 mile long; wheels, two units, each a 40-in. single-overhung Doble impulse, 1800 hp., 514 r.p.m., about 1130-ft. head.

11. Verdi, 1912. Total length conduit, 2.1 miles, with 2.0-mile canal and 0.1-mile timber flume 10.5 by 6.5 ft.; small timber forebay; penstock 0.5 mile long, two-thirds 108-in. wood-stave and one-third 72-in. steel pipe; wheels, one 45-in. horizontal, 3200 hp., 277 r.p.m., 92-ft. head.

12. Washoe, 1902. Total length conduit, 2.1 miles, with 1.4-mile canal and 0.7-mile 10- by 6½-ft. wood flume; small forebay; penstocks two 72-in. wood-stave pipe; wheels, two 30-in. horizontal, about 1250 hp., 360 r.p.m., 86-ft. head.

13. Folsom, 1895. Total length canal, 1.7 miles—largest power canal in California; small forebay; penstocks four 96-in. steel pipes; wheels, four 30-in. horizontal, 1100 hp., 300 r.p.m., 55-ft. head.

14. Seros, 1913. Total length of waterway, 17.6 miles, with 11.8-mile upper canal as described, several storage reservoirs (2230 acre-ft. total) and lower canal 2.6 miles.

FLUMES

Open flumes are often used alone or in connection with canals, to avoid detours which would be necessary for the canal in following the contour of a side hill and often in directly crossing some valley or ravine. They may be constructed of wood, steel, or reinforced concrete and may

usually be materially less in cross-sectional area than the canal, due to less frictional resistance and, therefore, greater allowable velocities. Care must be taken to construct gradual transition sections (usually of concrete) between the canal and any flumes at both entrance to and exit from the latter, to avoid undue loss of head in this transition.

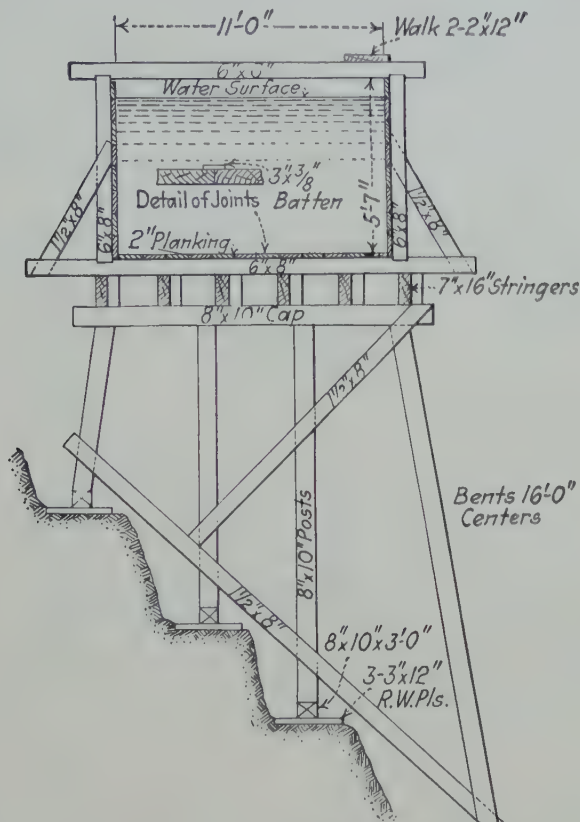


FIG. 134.—Wooden flume—capacity 350 sec.-ft. ($n = 0.013$, $s = 0.0005$).

Wooden Flumes.—These are normally rectangular in cross section; for best hydraulic section the width is twice the water depth. A design for such a flume of 350 sec.-ft. capacity is shown in Fig. 134.¹

Wood-stave flumes, approximately semicircular in form, are also in general use, in diameters from about 2 to 20 ft. They are similar in construction to wood-stave pipes, the bends terminating in washers and nuts bearing upon cross-spreader timbers, one for each band.

Wooden flumes depreciate rapidly and are relatively short lived. They have high hydraulic efficiency, the value of n for such a flume as shown in Fig. 134 being about 0.013.

¹ *Trans. A.S.C.E.*, vol. 79, p. 1013, 1915.

Metal flumes of the non-riveted metal type have been used since about 1902. The earliest flume of this type—the Maginnis—is shown in Fig. 135.¹ The two overlapping sheets *a* and *b* are held between the rod *c* and the compression bar or curved channel *d*, in compression, and the flume sheets pressed together. Rod *c* carries the weight of flume and water through the carrier beam *g* to the stringers *h*. The Maginnis flume has a relatively rough interior, which reduces its carrying capacity, and this has resulted in the development of various different types of joints—the Newcomb, American, Thompson, etc.—all intended to give a relatively smooth flow line. Standard sizes of flumes are determined by stock sizes of sheets from which they are made and are designated commercially by

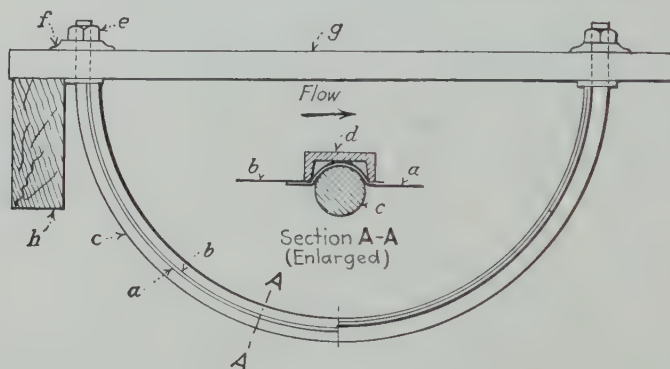


FIG. 135.—Maginnis metal flume.

“numbers” which are equal to the perimeter of the semicircle in inches. These sheets may be of any suitable metal; if of steel or iron, they should be galvanized. The desirable thickness of steel sheets according to the Reclamation Service requirements is:

Numbers	Gage
Up to 48.....	22
48- 96.....	20
96-132.....	18
132-168.....	16
168-204.....	14
204-252.....	12

Experimental values of *n* for semicircular flumes as determined by the Reclamation Service are shown at the top of page 347.

The cost of erection of the flume barrel will be roughly about 10 per cent of the materials, not including freight and hauling. The field cost of metal work in place as constructed by the U. S. Reclamation Service in 1919-1920 was from 15 to 20 cts. per pound.

¹ HINDS, JULIAN, “The Design, Construction, and Use of Metal Flumes,” *Eng. News-Record*, May 25, 1922, pp. 854-861.

Type of joint	Range of n		
	Mean	Maximum	Minimum
Lennon.....	0.0111	0.0130	0.0087
Smooth.....	0.0123	0.0131	0.0103
Maginnis.....	0.0185	0.0175	0.0202
Newcomb.....	0.0128	0.0123	0.0135

Concrete Flumes.—A recent type of concrete-flume construction is shown in Fig. 136.¹ This was built in 1923 by the San Joaquin Light and Power Company on the North Fork of the San Joaquin River, California, to replace a ditch and wooden flume constructed in 1910.

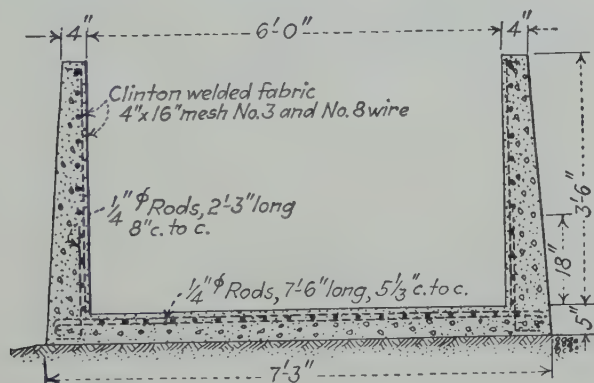


Fig. 136.—Concrete flume—San Joaquin Light and Power Company.

It is $2\frac{1}{2}$ miles long, with a typical section as shown, and has the following hydraulic properties:

Slope, 1 in 675, $V = 5.54$, $Q = 100$, $n = 0.014$. It was constructed in 8 months.

A precast-concrete flume built by the U. S. Reclamation Service on the Klamath project² is illustrated in Figs. 137 and 138. This is 4316 ft. long, including inlet and outlet transitions, with a capacity of 335 sec.-ft. The flume units are 11 ft. wide, 6 ft. high, and 4 ft. 3 in. long, as shown in Fig. 137, and are supported on solid bents varying from 1 to 5 ft. in height and framed bents from $5\frac{1}{2}$ to $15\frac{1}{2}$ ft. high. The highest bent is shown in detail in Fig. 138.

The precast work included 2600 cu. yd. at a unit cost of \$46.80 or a total of about \$121,700. The entire cost, including inlet and outlet structures, superintendence, and general expense, was about \$198,300 or

¹ *Eng. News-Record*, July 5, 1923, pp. 20–21.

² *Eng. News-Record*, Feb. 1, 1923, pp. 194–199.

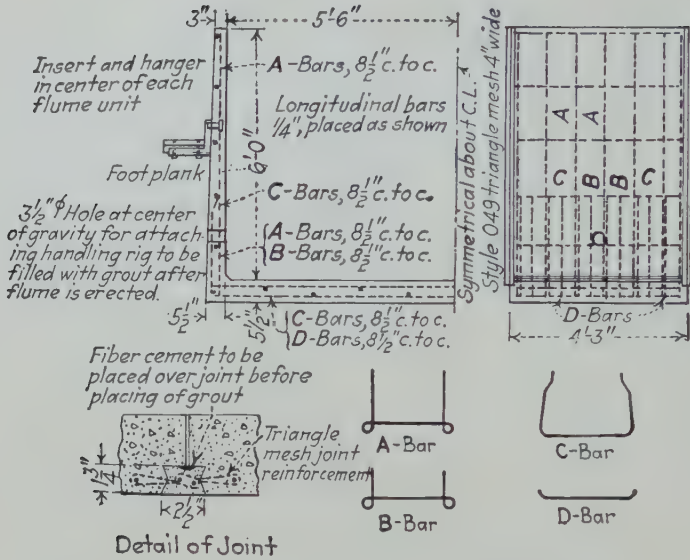


FIG. 137.—Precast concrete flume—U. S. Reclamation Service.

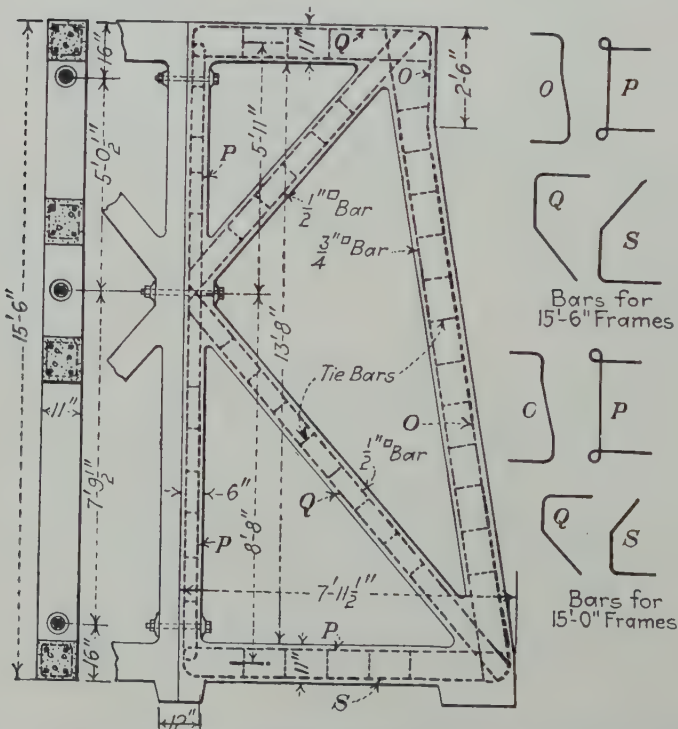


FIG. 138.—Details of bent for precast concrete flume—U. S. Reclamation Service.

about \$46 per linear foot. Studies made prior to construction showed relative costs, including maintenance and repairs, as follows:

	Per Cent
Precast concrete.....	100
Semicircular galvanized metal.....	137
Semicircular wood.....	116
Monolithic concrete.....	137
Lock-joint pipe with cradle.....	124
Lock-joint pipe without cradle.....	126

It should be noted, however, that the actual cost of the precast-concrete construction materially exceeded the amount of these preliminary estimates.

The use of concrete for flumes is likely to increase, due to their lower cost of maintenance and longer life, as compared with wooden flumes. The life of a concrete flume is not yet established, and much evidently depends upon the quality of the concrete and care taken in construction.

TUNNELS

Waterways are often constructed as tunnels to save distance, as in cutting across a bend in a river and where the topography prevents the use of a canal or pipe line in using such a direct line. The best material for tunnel construction is solid rock with little ground water; the poorest, unstable earth or unstable rock with much ground water. Where solid rock is assured, the cost of tunnel construction may be estimated with reasonable accuracy. In poor material, especially where water is likely to be encountered, the cost is likely to be very uncertain.

The necessary cross section of a power tunnel, as a minimum, will be that permitting construction, or say a section about 5 to 6 ft. high and 6 to 8 ft. wide, or 30 to 40 sq. ft. of area, corresponding to a discharge capacity of perhaps 200 to 250 sec.-ft. Above about this minimum amount, the section will be determined by the discharge to be handled; the best theoretical section, as in the case of other types of waterways, is that which will give a minimum annual cost of tunnel and power lost by friction.

Plant 4 of the New England Power Company, at Shelburne Falls, Mass., constructed in 1913, has a waterway of the tunnel type, shown in typical cross section in Fig. 139. This is about 1600 ft. long, all in rock but about 200 ft. at the lower end. The three wheel units at the plant (each 3200 hp., 297 r.p.m., 64-ft. head) use at full gate about 1100 sec.-ft., requiring, therefore, a maximum velocity of about 8 ft. per second, for which the hydraulic slope is about 0.0015, and total loss of head about 2.4 ft., or about 3.8 per cent of the total head utilized, of 64 ft. The average loss of head in the tunnel is probably not over half this amount.

A more recent tunnel waterway of this company is at the Davis Bridge plant (1924), shown in cross section in Fig. 140¹ (see also Chap. XIII, page 697). This is 12,780 ft. long, in hard tough mica schist. Studies indicated that for the necessary capacity a lined tunnel of a smaller section was more economical than a larger unlined section. The section

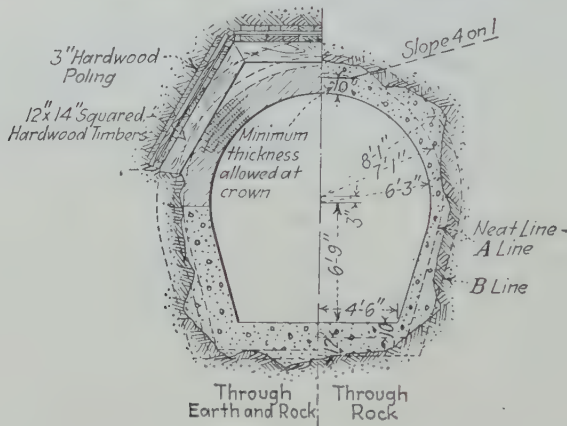


FIG. 139.—Tunnel sections—Plant 4, New England Power Company.

adopted provided maximum working width at the bottom, for excavation operations, and a semicircular top to give facilities for placing the lining.

The three wheel units (each 20,000 hp., 360 r.p.m., 350-ft. head) use at the full-load point about 1400 sec.-ft.

or a tunnel velocity of about 8.5 ft. per second, and total loss of head of about 16.0 ft. Steel penstocks 9 ft. in diameter and about 620 ft. long serve each unit, causing a further loss of head of about 2.4 ft., or a total for the whole waterway of about 18.4 ft., about 5 per cent of the total operating head. Under average conditions this is, of course, materially reduced.

24.5 Sq. Ft. Overbreak— $5\frac{1}{2}$ " Average

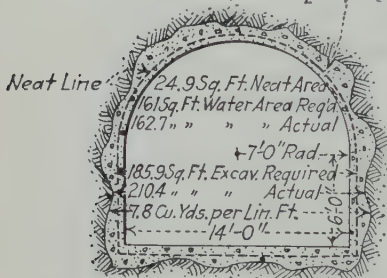


FIG. 140.—Section of power tunnel—Davis Bridge development.

The cost of this tunnel was about \$1,750,000, or \$136 per linear foot.

It averaged 7.8 cu. yd. excavation per linear foot, or about \$17.50 per cubic yard total cost.

Tunnel 1 of the Caribou development, North Fork, Feather River, California, is an instance of difficult construction conditions. This is 11,200 ft. long of varying section, approximately rectangular, with a water area of about 45 sq. ft. (about 7 ft. high by $6\frac{1}{2}$ ft. wide), constructed

¹ EATON, A. C., "Davis Bridge Development," *Jour. Boston Soc. Civil Eng.*, January, 1925.

in lava formation and mostly timber lined. Much earth and water were encountered, requiring pumping of more than 15 sec.-ft. for long periods. It cost about \$200 per linear foot, averaging about 4 cu. yd. per linear foot or some \$50 per cubic yard total cost.

A tunnel (1924) in Pennsylvania of "workable size"—5½ ft. high by 7 ft. wide by 5700 ft. long, mostly in hard traprock and unlined (part of 36-in. pipe line)—cost about \$39 per foot, or about \$27.50 per cubic yard.

In considering the use of a tunnel waterway, therefore, unless the material is obviously rock, cost estimates will require the best judgment of engineers experienced in such work and may even then be subject to much uncertainty.

In preliminary estimates of rock-tunnel cost, where conditions are not unusual, the cost per linear foot per square foot of water area is a convenient unit. Based upon a 200 per cent cost index (see Table 122, Chap. XII), the following data for concrete-lined tunnels may be used:

Water Area, Square Feet	Cost per Linear Foot of Tunnel per Square Foot of Water Area
50	\$1.20
100	1.00
150	0.80
200	0.60

PENSTOCKS

The term penstock as ordinarily used refers to a relatively short stretch of pipe or pipes near the power house connecting the wheel units with the main waterway, the latter often being a canal or open channel terminating in an enlargement or forebay from which the penstocks take off. In the case of a short penstock, there would normally be a pipe for each wheel unit.

Where the main waterway is a pipe or conduit rather than an open channel—this often requiring a considerable length—the terms pipe line and flow line, as well as penstock, are used. To distinguish between pipe lines used for water power development and those for other purposes, such as water supply, however, it seems appropriate to use the term penstock as applying to any pipe or pipes between intake at the dam and the power house of a water power development.¹

Penstocks may be of steel in various forms, reinforced concrete, wood stave, or combinations of these types; and occasionally for small developments cast-iron pipe may be used.

Location.—In the location of a penstock the "economically shortest" route is desired, as in the case of a canal. The grade of a penstock line is not limited, as with the canal (which must be nearly level), as the pipe

¹ This is in accord with the definition in Webster's "New International Dictionary," viz., "a closed-conduit tube or pipe for conducting water, as to a waterwheel."

need in general only be kept below the hydraulic gradient. Note that this is the maximum hydraulic gradient, or that corresponding to full plant or peak capacity, at low-head water level. It is usually desirable to have the penstock always sloping toward the power house, but its grade may be varied as desired to fit the topography. On the other hand, the more economical location would prevail where the penstock can be kept on as flat a grade as possible, consistent with keeping below the hydraulic gradient, for the greater part of its length—in order to minimize pressures and hence cost of pipe—with a sudden pitch to the power house through a relatively short distance.



FIG. 141.—Wooden trestle for wood-stave penstock—Searsburg development.

Occasionally, for economy, it may be necessary to use a siphon (or more properly an inverted siphon) where a deep valley has to be crossed, although this should be avoided if possible, as it complicates the matter of emptying the penstock, and there is some tendency for air to collect just below the siphon, requiring air valves and some extra attention in operating. The cost of pipe for the siphon will also be greater on account of the greater pressure. The saving in length of line by an inverted siphon may often warrant its use, however, as in the case of the Medbury Brook siphon on the 8-ft. wood-stave penstock of the Searsburg development of the New England Power Company (see Fig. 292), where the line sags about 65 ft. in elevation below the general grade.

Wooden or reinforced-concrete trestles to support the penstock may sometimes be used in lieu of earth fills, where a short valley must be crossed, as shown in Fig. 141 for the Searsburg development.

The intake of the penstock at the dam or forebay of the canal must be at a level low enough to provide an adequate water seal under all conditions, particularly at low water. This will commonly mean that the top of the penstock at its intake should be 4 or 5 ft. or more below the lowest water level contemplated at the dam or in the forebay, the desirable amount of water seal varying with the size of penstock and velocity of flow. If there is too little depth of cover, whirlpools will tend to form and carry air into the penstock and to the wheels, tending to lessen the power output.

A gate and usually racks are placed at the entrance to the penstock. Below the gate and near it should be an air vent or standpipe connecting the top of the penstock with the open air and running to a level above the head water. This is to permit air to enter the penstock when the head gates are closed and water drawn off through the wheels, otherwise a dangerous collapsing pressure may be exerted on the penstock. Care must also be taken in operation to see that the water in the vent pipe does not become frozen, thus preventing the entrance of air.

In addition to proper depth of water seal, the entrance to the penstock should be flared to avoid any loss of head by contraction.

Sharp bends in the penstock line should be avoided, as far as possible, as they cause loss of head and require special anchorages.

At the power house, where ordinarily a steel penstock would be used, connections with the wheel units may be in various ways. Where individual penstock lines run from forebay to power house, the typical vertical setting of wheel unit with scroll casing is conveniently adapted for direct connection at the level of the wheel casing (or somewhat below the floor level). In this case the wheel units are in the line of the greatest dimension of the power house, with penstock lines normal thereto, spaced to fit the distance between wheel units.

Where the waterway is a single penstock, it must branch near the power house to give the individual unit connections. This may be effected in various ways. The main penstock may terminate in a large tee, with branch the size of the penstock at right angles to it, from which the individual pipes to the units take off. This arrangement is simple to fabricate but results in loss of head by expansion where the main penstock enters the tee branch and again by contraction at entrance to the smaller pipes.

A better form of connection is the Y-branch, using a single Y for two units; a Y with a large and a small branch, the large connecting with a second Y, for three units; two Y's for four units; etc. This type of connection is shown in Fig. 142, which is a shop assembly of a 48- by 24-in. Y-branch. The arrangement of sections of pipe and type of joints should be especially noted. The two field joints are the highest

girth joints not riveted. By this arrangement sudden changes in velocity and loss of head may be avoided.

An interesting arrangement of penstocks and power-house location¹ is shown in Fig. 143 for the Fagundes plant of the Cia. Brasileira de Energia Electrica, located about 70 miles north of Rio de Janeiro and completed in 1924. The main penstock line is of riveted steel 66 in. in diameter and 5070 ft. long leading from a 40-ft. storage dam through a 665-ft. tunnel 6.5 ft. wide by 11.75 ft. high; thence, as an inverted siphon,

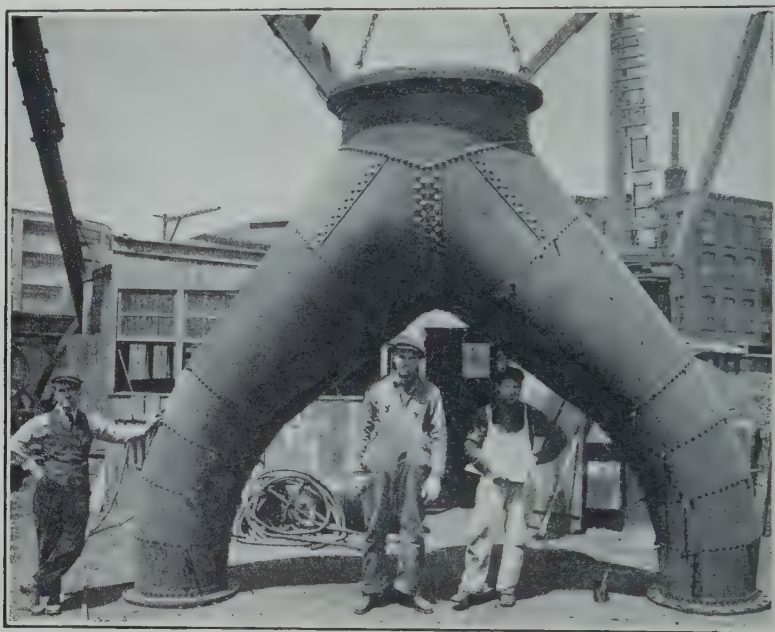


Fig. 142.—Y branch for steel penstock. (Courtesy of Walsh's Holyoke Steam Boiler Works.)

over the river on a novel type of reinforced-concrete arch-suspension bridge, 75 ft. long, shown in Fig. 144. At this point $\frac{9}{16}$ -in. plate is used.

The main penstock terminates in an 86-in. distributor pipe, encased in masonry, at a standpipe or surge tank 280 ft. long and 86 in. in diameter, resting upon the side hill. An auxiliary standpipe of the same diameter as the penstock and 207 ft. long is placed about halfway to the river crossing and may also be seen at the extreme left in Fig. 143. The individual penstock lines to the two wheel units are 48-in. steel pipes, each 730 ft long, with venturi meters, for flow measurement, at the lower end.

A main control valve, with electric remote control from the power house, is placed at the distributor end of the 66-in. penstock line.

¹ Data furnished by Pelton Water Wheel Company of New York.



FIG. 143.—Fagundes development of Cia Brasileira de Energia Electrica near Rio de Janeiro. (Courtesy of Pelton Water Wheel Company.)



FIG. 144.—Fagundes development concrete "arch-suspension" bridge for 5½-ft. steel penstock. (Courtesy of Pelton Water Wheel Company.)

The wheel units are horizontal scroll-case Francis type, each 4000 hp., operating at 376-ft. head, with a speed of 600 r.p.m., made by the Pelton Water Wheel Company of New York. Each turbine has two bearings with generator rotor between and exciter rotor overhung on the end of the shaft beyond the outboard bearing.

Other features of interest in Fig. 143 are the penstock anchorages and piers, arrangement of power house, tailrace, etc. The power house is 72 by 57 by 40 ft. high, constructed of reinforced concrete, and is especially well ventilated and lighted.

Economic Size of Penstock.—As already noted, the fundamental basis for best size of penstock is that where the annual cost (*i.e.*, of penstock plus value of power lost in friction) is a minimum, the second factor being computed for average load. The solution for best size of penstock can generally be made best by plotting curves of total annual cost against size of pipe in a similar manner to the method previously described under canals. This method is advantageous in that any type of pipe may be considered and actual costs for a particular location used, thus taking into account any special conditions which may prevail.

For steel pipe, or any material where the cost may be expressed as a function of its weight, formulas may be derived giving the best size of pipe directly in the following manner:¹

Assuming:

P = loss in horse power per linear foot of pipe due to friction head hf in penstock.

Q , A , and V = average discharge, area, and mean velocity for penstock.

d = diameter of pipe, feet.

f = friction factor.

S = allowable stress on gross section of pipe, pounds per square inch.

c = cost of pipe per pound.

C = cost of pipe per linear foot.

b = value of 1 hp.-year of power (theoretical horse power).

i = yearly fixed charges on pipe line expressed as a ratio.

L = total annual cost per foot of penstock, dollars.

$$P = \frac{Qh_f}{8.8} = \frac{Q f v^2}{8.8 d 2g} = \frac{Q f}{8.8 d} \frac{Q^2}{2g \times \left(\frac{\pi d^2}{4}\right)^2} = 0.00287 \frac{f Q^3}{d^5}$$

$$C = \frac{0.434h \cdot d \cdot 12}{2S} \times \frac{\pi d}{12} \times 490 \times c = 333 \cdot \frac{chd^2}{S}$$

$$L = Ci + Pb$$

$$= \frac{333chid^2}{S} + 0.00287 \frac{fbQ^3}{d^5}$$

¹ MEAD'S "Water Power Engineering," Par. 263.

$$\begin{aligned}\frac{dL}{dd} &= \frac{666chid}{S} - 0.01435 \frac{fbQ^3}{d^6} = 0 \\ 666chid^7 &= 0.01435fbQ^3S \\ d^7 &= 0.00002154 \frac{fbQ^3S}{cih} \\ d &= 0.215 \sqrt[7]{\frac{fbQ^3S}{cih}}\end{aligned}\quad (3)$$

It is well to note the effect upon d in the above formula due to variation in the factors under the radical. For illustration, doubling any one of these factors other than Q affects d as $\sqrt[7]{2} = 1.10$ or about 10 per cent, a relatively small change in diameter. Hence, if these factors are known approximately, this referring particularly to b , which is usually least well known, the best value of d can be determined with fair accuracy.

If Q is doubled in the formula, it would increase the pipe diameter in the ratio of $\sqrt[7]{(2)^3} = 1.34$ or add about one-third to the diameter.

For a penstock under a low head, such that a minimum thickness of pipe t^1 must be used, the formula will be changed as follows:

$$\begin{aligned}C &= \frac{\pi dt^1c}{12} \times 490 = 128.3dt^1c \\ L &= 128.3dt^1ci + 0.00287 \frac{fbQ^3}{d^5} \\ \frac{dL}{dD} &= 128.3t^1ci - 0.01435 \frac{fbQ^3}{d^6} \\ d^6 &= 0.000112 \frac{fbQ^3}{t^1ci} \\ d &= 0.219 \sqrt[6]{\frac{fbQ^3}{t^1ci}}\end{aligned}\quad (4)$$

Ordinary values for the factors in these two equations are as follows:

$f = 0.02$ to 0.03 .

$S = 8000$ to $10,000$ lb. per square inch.

$b = \$5$ to $\$25$ per horse-power-year.

$c = 6$ to 9 cts. per pound.

$i = 0.10$ to 0.12 .

As previously stated, b is the value of the water power per horse-power-year (of theoretical power as used in the formulas), which is in turn the difference between its cost (not including cost of penstock) and its sale price. This margin of value may vary, depending on the value of the privilege, from $\frac{1}{10}$ to $\frac{1}{2}$ ct. per kilowatt-hour at switchboard. The corresponding range in b is from $\$0.001 \times 0.746 \times 0.80 \times 8760 = \5.20 to $\$26$ per horse-power-year.

These formulas may be used in any case where the cost of pipe in place can be expressed as a function of its weight. For steel pipe this would not include cost of grading, of cradles, anchorages, etc. On the other hand, the greater part of the cost is of the pipe, including its laying, so that the formula often gives quite closely the best size of steel pipe. Furthermore, as concrete and wood-stave pipes usually do not vary greatly in cost from steel pipe, the formula may be conveniently used to approximate the best size of any penstock, whether steel or other material.

Adams¹ deduced for steel penstocks the rule: "That pipe fulfills the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals 0.4 of the annual cost of pipe line." This will be found to be approximately true by substituting the value of d as found in Eq. (3) in the equation for L , which will result in a ratio of Pb/Ci of 0.41.

O. L. Hooper has derived a formula for the best size of concrete penstocks as follows:

$$d = 0.215 \sqrt[7]{\frac{fbQ^3}{i \left[0.0000290a + 2.61pc + \frac{ch}{S} \right]}} \quad (5)$$

where notation is as for Eq. (3), and

a = cost of concrete per cubic yard.

p = ratio of the cross-sectional area of the longitudinal steel to that of the concrete.

c = cost of steel reinforcement, dollars per pound.

S = allowable stress in the hoop steel, pounds per square inch.

This formula is similar to Eq. (3) with the exception of the first two terms in the parenthesis in the denominator.

H. D. Gurney² has also derived a formula similar to Eq. (3) for best size of wood-stave penstocks, as follows:

$$d = 0.215 \sqrt[7]{\frac{fbQ^3S}{ih(c + 0.000016WS)}} \quad (6)$$

where, in addition to notation as in (3),

c = cost of steel bands, dollars per pound.

W = cost of wood staves, dollars per board foot.

Penstock Velocities.—It should be kept in mind that the best size of penstock is based upon the loss of head at average load, and the corresponding velocities are average velocities. Thus, for an assumed case, where

¹ *Trans. A.S.C.E.*, December, 1907, p. 173 *et seq.*

² Thesis, Massachusetts Institute of Technology, 1932.

$Q = 250, f = 0.02, S = 8000, b = \$15, c = 5 \text{ cts.}, i = 0.10, h = 110,$

$$d = 0.215 \sqrt[7]{\frac{0.02 \times 15 \times 8000 \times 250^3}{0.05 \times 0.10 \times 110}} = 7.60$$

The corresponding area of pipe is about 45.2 sq. ft. and mean velocity 5.5 ft. per second. The maximum velocity would be 5.5/capacity factor, or say 11 ft. per second for a 50 per cent capacity factor, the hydraulic gradient for which at low-head water level would be of importance in locating the penstock line, as previously noted.

The average loss of head would be more strictly the average of the losses at the different operating wheel gates rather than the loss corresponding to average velocity, as the losses vary as $V^2/2g$, but to determine accurately the average losses in this manner would involve much detailed computation. With a 50 per cent capacity factor, while the average load is 50 per cent of the plant capacity, the average wheel-gate opening is greater than that corresponding to 50 per cent of plant capacity, due to the use of several wheel units and the operation of these as near to best efficiency as practicable. At half load, for example, with three wheel units, two of the units may be carrying the load—each at 0.6 or 0.7 gate; at one-third load, one unit may be carrying the load at, say, 0.8 gate.

The average wheel-unit load for a 50 per cent capacity factor may, therefore, be about 0.7 of the wheel capacity and the average velocity head about 0.5 that at full gate capacity. Hence, the use of the average wheel discharge or wheel capacity $\times$ capacity factor as a basis for penstock size probably gives approximately correct results in most cases.

In long penstock lines, however, the losses at full capacity must be kept in mind, and a somewhat larger size of penstock may be required in order not to curtail plant capacity by excessive loss of head, with the plant fully loaded.

For moderate heads, maximum velocities will ordinarily be limited to 10 to 15 ft. per second; for higher heads, velocities up to 20 ft. per second are not uncommon.

Another factor to be kept in mind in fixing penstock velocities is speed regulation or control of the turbines, which with very high velocities is more difficult. Higher velocities may be used with impulse and scroll-case wheels than with a cylindrical-case wheel.

In Table 69 data relating to penstock velocities for some recent plants are given.

A study of data in this table indicates that maximum penstock velocity (for full wheel discharge) commonly is from 0.05 to 0.07 $\sqrt{2gh}$ (where h is the effective head on the plant) for reaction wheels and heads up to 500 or 600 ft. For impulse wheels and the upper range of reaction-wheel plants above about 500-ft. head, v may be as low as 0.02 to 0.03 $\sqrt{2gh}$.

TABLE 69.—VELOCITIES IN PENSTOCKS AS CONSTRUCTED

Number	Plant	Location	Date built	Wheel capacity, horse power	Number of units	Head, feet	Maximum discharge, cfs	Penstock			$\sqrt{2gh}$	$\frac{v}{\sqrt{2gh}}$	Remarks
								Kind	Length, feet	Diameter	Maximum velocity, feet per second		
1	Searsburg, New England Power Company	Deerfield River, Vermont	1921	6,000	1	230	300	Wood stave	18,500	8.0'	6.0	122	0.05
2	Davis Bridge, New England Power Company	Deerfield River, Vermont	1924	60,000	3	350	1,770	Steel	13,000	6.5'	9.0	150	0.07
3	Plant 5, New England Power Company	Deerfield River, Vermont	1914	24,000	3	240	970	Concrete	5,000 ±	3 @ 9.0'	9.3	124	0.06
4	Minnewawa Brook, Keene Gas and Electric Company	Marlboro, N. H.	1923	2,500	2	270	100	Steel	400 ±	3 @ 7.0'	8.5	...	0.07
5	Pit River 1, Pacific Gas and Electric Company	California	1922	80,000	2	454	1,820	Wood stave	5,750	4.0'	8.0	132	0.06
6	San Francisco 2,	City of Los Angeles	1920	61,500	2	515	1,320	Steel	31,372	9.0' ±	28.6	170	0.17
7	Caribou, Great Western Power Company	California	1921	90,000	3	1,008	800	Steel	11,430	110' ±	12.0	182	0.07
8	Sherman Island, International Paper Company	Hudson River, New York	1923	50,000	5	66	7,500	Tunnel	1,200	3 @ 7.0'	11.5	253	0.06
9	Big Creek 1, Southern California Edison Company	California	1913	40,000	4	2,100	200 ±	Tunnel	2,755	40' ±	17.5	...	0.07
10	Mystic Lake,	Montana	1925	15,000	2	1,050	150	Concrete	3,372	3.5'	8.1	65	0.03
11	Kern River 3,	California	1921	45,000	2	800	600	etc.	176	15 @ 10.5'	4.7	...	0.07
12	Bartlett's Ferry,	Chattahoochee River, Georgia	1925	22,000	2	112	2,040	Steel	3,880	12'	1.8	318	0.02
13	Tallulah Falls,	Tallulah River, Georgia	1913	85,000	5	580	1,600	Steel	9,000	4 @ 3.67'	4.7 to 7.0	...	0.02
14	Hiram, Maine,	Saco River, Maine	1916	3,750	1	75	580	Wood stave	2,730	4.67'	8.8	260	0.03
15	Lower Fifteen Mile Falls	Connecticut River, New Hampshire	1931	215,000	4	170	12,000	Tunnels, etc.	68,000	4.00'	12.0	227	0.05
16	Sherman,	Deerfield River, Massachusetts	1926	10,000	1	78	1,400	Steel	2,520	2 @ 7' to 5'	7.7 to 15.2	...	0.03 to 0.07
									300	15'	11.5	85	0.13
									6,666	151' ±	10.6	193	0.07
									1,230	5 @ 5'	16.5	70	0.08
									460	11'	6.1	105	0.09
									150	19.0'	10.5	71	0.10
									235	13.0'	10.5	...	0.15

The length of penstock is also a factor affecting velocity. Where the penstock is short as may be noted in Table 69, higher velocities are allowable.

STEEL PENSTOCKS

For pressures above about 150 ft. of head, steel is usually the most satisfactory material for penstocks. Steel pipe may be (1) riveted, (2) welded, (3) lock bar, (4) spiral riveted.

Riveted-steel Penstocks.—The thickness of a steel pipe is determined by the usual hoop-tension formula

$$t \text{ (inches)} = \frac{0.434h \times d}{2f} \quad (7)$$

h being in feet, d in inches, and f in pounds per square inch, the latter being the allowable tensile strength of the plate (12,000 to 15,000 lb. per square inch) multiplied by the joint efficiency. The latter may vary from 50 to 90 per cent, depending on the type of joint, so that the allowable f in the formula may actually be (on the basis of 15,000 lb. per square inch plate tension) from about 7500 to 13,000 lb.

For various practical reasons of field handling, avoiding distortion of pipe, allowance for corrosion, etc., steel pipe must be of a certain minimum thickness, depending on its size, good practice being about as follows:

Diameter, Feet	Minimum Thick- ness, Inches
6	$\frac{1}{4}$
7	$\frac{5}{16}$
10	$\frac{7}{16}$

This results in a considerable waste of metal and, hence, lack of economy in using steel pipe under moderate pressures. Thus, with net f as 10,000 lb., the thickness as given in the previous table would withstand hoop tension for heads of 160, 170, and 170 ft., respectively. To allow for water hammer, these heads should be reduced somewhat, approximately to 150 ft. For all heads below about 150 ft., therefore, the steel pipe as practically built is wasteful of material.

For very large steel penstocks, say, over 15 ft. in diameter, it is desirable to use stiffening rings, usually of angles, to prevent distortion and tendency for seams to open.

Usually, to allow for corrosion, $\frac{1}{16}$ in. is also added to the computed thickness of pipe.

The largest riveted-steel penstock in use in this country appears to be that at the Bridgewater, N. C., development, which is 20 ft. in diameter and 800 ft. long.

General Arrangement.—The longitudinal seams which carry the hoop tension may be single- or double-riveted lap, or double-, triple-, etc., riveted butt joints. With low heads the longitudinal seams, if tight, will have an excess strength. When practicable, it is advantageous to make the pitch of the circular and longitudinal seams the same, to save cost, even if in doing this rivets are added to the longitudinal seams. The lap joint is easy to make, but, above about a $\frac{1}{2}$ -in. plate, it is hard to get a good job and for thicker plates the double-butt joint is stronger and more satisfactory. A single-butt joint (single strap) has the same strength as a lap joint but is more expensive than the latter, and, hence, is not desirable for longitudinal seams.

Details of standard riveted joints of the Pacific Coast Electrical Association are given in Table 70,¹ including the range in use of different types of joints and their efficiency.

For the circular or girth joints, single- or double-riveted lap joints are used. The single-butt joint makes less obstruction to flow but as just noted is more expensive than the lap joint and no stronger.

TABLE 70.—TYPES OF JOINTS AND EFFICIENCIES, ETC.

Type of joint	Range in plate thickness, inches	Joint efficiency, per cent		Range in ratio of rivet diameter to plate thickness
		Range	Average	
Single-riveted lap.....	$\frac{1}{4}$ – $\frac{5}{8}$	59–47	50	3–2
Double-riveted lap.....	$\frac{1}{4}$ – $\frac{3}{4}$	71–69	70	3–1.67
Triple-riveted lap.....	$\frac{1}{4}$ –1	75–71	73	2–1.25
Double-riveted double-butt.....	$\frac{3}{8}$ –1	83–82	83	2–1.25
Triple-riveted double-butt.....	$\frac{3}{8}$ –1 $\frac{1}{4}$	88–82	85	1.67–1.0
Quadruple-riveted double-butt.....	$\frac{3}{8}$ –1 $\frac{1}{4}$	94–85	90	1.67–1.0

As will be noted, the range in joint efficiency is from 50 per cent or less for single-riveted lap joints, to 90 per cent or more for quadruple-riveted double-butt joints. Note, also, that butt joints of a given kind of riveting, as double, triple, etc., are 12 to 13 per cent higher in efficiency than corresponding types of lap joints. Double-riveting is materially more efficient than single-riveting (20 per cent for lap joints), but triple-riveting adds but slightly to the joint efficiency.

Rivet size varies with type of joint and thickness of plate, the range being given in the previous table. As will be noted, this is greater for lap joints, ranging from 3 to 1.25 times plate thickness, while with butt joints this range is from 2.0 to 1.0.

¹ *Report of Hydraulic Power Committee, Nat. Electric Light Assoc., September, 1923, pp. 32–38.*

Another rule sometimes used is:

For lap joints, $d = 2t$, but not $< \frac{5}{8}$ in. or $> 1\frac{1}{8} \pm$ in.

For butt joints, $d = 1\frac{1}{3}t \pm$.

Detailed specifications for riveted-steel pipe are given in the National Electric Light Association *Report* referred to above. Certain of the more important details are as follows:

General Specifications:

Length of Sections:

In general, riveted pipe is made in three- to four-course sections, each section totaling 18 to 32 ft.

Joints:

Details as per standards.

Circular lap joints with larger size up hill.

Longitudinal lap joints to point down, located alternately 30 deg. to right and left of top c/L of pipe.

Longitudinal butt joints at top c/L of pipe except where angle sections, air valves, manholes, etc., occur, in which case locate near horizontal diameter.

Angle Sections:

At angles or curves in plan or grade, plates cut and punched to form a small oblique angle at circular seams, as many courses as necessary.

Deflection angle between two courses 1 to 5 deg., usually not less than three courses in an angle section.

Longitudinal seams near and a little above horizontal diameter line.

Punching and Reaming:

Rivet holes for butt joints and lap joints for plate over $\frac{3}{8}$ in. to be subpunched $\frac{1}{8}$ in. and reamed to size.

Diameter of rivet hole $\frac{1}{16}$ in. greater than rivet diameter.

Rolling:

Plates bent cold to true circular shape.

Assumptions Used in Design.

Plate, standard A.S.T.M. for boiler flange steel.

Rivets, standard A.S.T.M. for boiler rivet steel.

Stresses (pounds per square inch):

Kind	Ultimate	Ordinarily used	Approximate ratio to tension
Tension.....	55,000	12,000-15,000	1
Shear:			
Single.....	44,000	9,000	$\frac{3}{4}$
Double.....	88,000	18,000	$1\frac{1}{2}$
Bearing.....	95,000	20,000	$1\frac{3}{4}$

Details:

Hole diameter $\frac{1}{16}$ in. larger than rivet diameter; use for rivet shear and bearing.
 Net Area.—For punched holes deduct $\frac{3}{16}$ in., for reamed holes $\frac{1}{16}$ in., on account of hole being greater than rivet.

Edge distances at least 1.5 times the diameter of hole.

Rivet Spacing.—Distance between runs such that sum of two net diagonal distances between holes not less than 1.25 times net distance between holes on gage lines. Maximum spacing along calked edges = $2\frac{1}{2}t + d + 1\frac{1}{2}$ in.

For the circular seams either lap or single-butt joints may be used, the latter being preferable hydraulically, but more expensive, usually adding from $\frac{3}{10}$ to $\frac{1}{2}$ ct. per pound to the cost of pipe. Ordinarily, single riveting will do for the circular joints, and it is desirable to make the number of rivets a multiple of 4, so that they will be at the 90-deg. points.

Rivets are driven either hot or cold, although the latter method is used on the lighter work.

Alignment.—The radii and spacing of curves should be selected so that, if practicable, full plates will come between angle points, as special

plates cost more and waste more metal. With this limitation, curves should not cost over one-third more than straight pipe.

In cutting plates for bends, it is desirable to have a uniform bevel, equal on adjacent plates, not greater than 2 deg. 30 min. or 5 deg. per joint in pipe. To get a short radius, both ends of the sheet may be beveled,

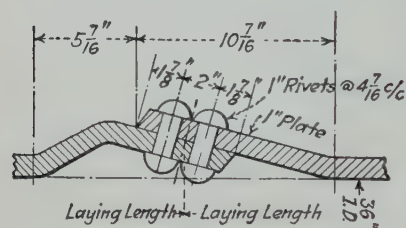


FIG. 145.—Bump joint—riveted steel pipe.

which, however, makes assembling more difficult. For less curvature, bevel every other sheet.

Note that the p.c. of the alignment curve will lie at the middle of the first section of beveled pipe (and similarly that the p. t. will be in the middle of the last beveled section) and that the curve will be tangent to the middle of each section.

Welded-steel Pipe.—Due to improvement in manufacturing methods, the use of forge-welded steel pipe for penstocks is rapidly increasing. Welded pipe is made in sizes from 12 in. up to any diameter that can be shipped, viz., 10 ft. or, in some cases, 11 ft. in diameter. Electric welding in making pipe and for joint connections is also making great progress.

The efficiency of the weld is usually taken at 90 per cent, based on manufacturer's guarantees, but many tests show practically 100 per cent efficiency. Welded pipe, therefore, is economical, saving the weight of butt straps, etc., and it is also smoother and more efficient hydraulically than riveted-steel pipe.

For welded pipe 26 in. and larger in diameter, the circular joints are often made of the "bump-joint" type (see Fig. 145), either single or double riveted. The bump joint is also of value in permitting slight deviations in alignment to maintain proper line but is more expensive than the lap joint. Below 26 in. in diameter, bump joints cannot be riveted and flange joints must be used.

Bends can be made readily with the bump joint, each joint as much as 5 deg. if necessary. Sharper bends are made from welded bends and tangents up to an angle of 14 to 22 deg. each or, where an anchor block is used, with a radius of not more than four or five diameters, so as to come within the limits of the anchor.

The minimum thickness recommended for welded pipe is as follows:

Diameter, Inches	Minimum Thickness, Inches
Up to 40.....	$\frac{1}{4}$
40-60.....	$\frac{3}{8}$
60 up.....	$\frac{1}{2}$

The maximum possible thickness of welded pipe depends upon the size of pipe, ranging from about $2\frac{3}{4}$ in. for very large pipes to half this amount for ordinary sizes. Thus, for the penstocks at Boulder dam¹ steel-welded pipe 30 ft. in diameter were made with a plate thickness of $2\frac{3}{4}$ in. These were made from plates 32 ft. long by 12 ft. wide and $2\frac{3}{4}$ in. thick, shipped to the job from the East. Three of these plates were shaped by a plate-bending roll and fabricated into a 12-ft. length of 30-ft. pipe and two such rings electrically welded together to form a single pipe section.

Banded Steel Pipe.—For high heads, where steel pipe of ordinary size would exceed about $1\frac{1}{4}$ in. in thickness, the requisite strength may be obtained by steel bands shrunk on to the pipe shell. The first use of this type of pipe in the United States was in 1927 at the Balch project of the San Joaquin Light and Power Corporation near Fresno, Calif., where the static head is 2381 ft.²

On this project the joint types were: Single-riveted bump joints in the upper end of the line, with double riveted when plate thickness reached $\frac{1}{2}$ in. Butt-strap joints, double riveted, begin in plate thickness of $\frac{6}{16}$ in. and continue to the Y, 300 ft. above the power house. Below the Y, only bolted flanged joints were used with $1\frac{3}{4}$ - to 3-in. bolts. The bands begin about midway of the penstock where the plate thickness of the unbanded pipe reached $1\frac{3}{16}$ in., the pipe shell being forge welded. The pipe sections are 20 ft. long for the entire length of the penstock. In the lower portions of the line the supporting piers were 40 ft. apart, with double-riveted butt-strap circular joints between the piers. In the upper part of the line, with larger diameter and thinner pipe shell, pier spacing was 20 ft.

Lock-bar Pipe.—This type of steel pipe, with longitudinal joints made by "lock bar" (closed to hold the plates by hydraulic pressure) (see Fig. 146), has been used extensively for pipe lines. This joint is

¹ *Eng. News-Record*, Dec. 21, 1933, p. 753.

² "Erecting a High-head Penstock on the Balch Project," *Eng. News-Record*, March 10, 1927, p. 406.

With coefficient of expansion of steel taken at 0.0000065 and $E = 30$ mill. lb. per square inch, with a temperature range of 50° , the corresponding stress per square inch if pipe is held rigidly will be

$$S = 50 \times 0.0000065 \times 30,000,000 = 9750 \text{ lb. per square inch}$$

requiring a double-riveted lap circular joint.

If in the above case the pipe is free to move, the linear expansion per 100 ft. of penstock would be about

$$50 \times 0.0000065 \times 100 = 0.03 \text{ ft. or about } \frac{3}{8} \text{ in.}$$

requiring an expansion joint perhaps every 300 or 400 ft., depending on the type of joint used.

Commonly, expansion joints will be placed about this distance apart unless anchor piers intervene, in which case there would be an expansion joint between each anchor pier, usually near and below the latter.

Anchorage for Steel Penstock Lines.—Anchorages will be required at angles in the pipe, whether in alignment or grade, and occasionally on long, steep grades to hold the pipe. Details of the penstock layout for

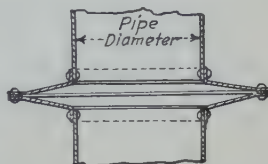


FIG. 148.—Diaphragm type of expansion joint for riveted steel pipe.

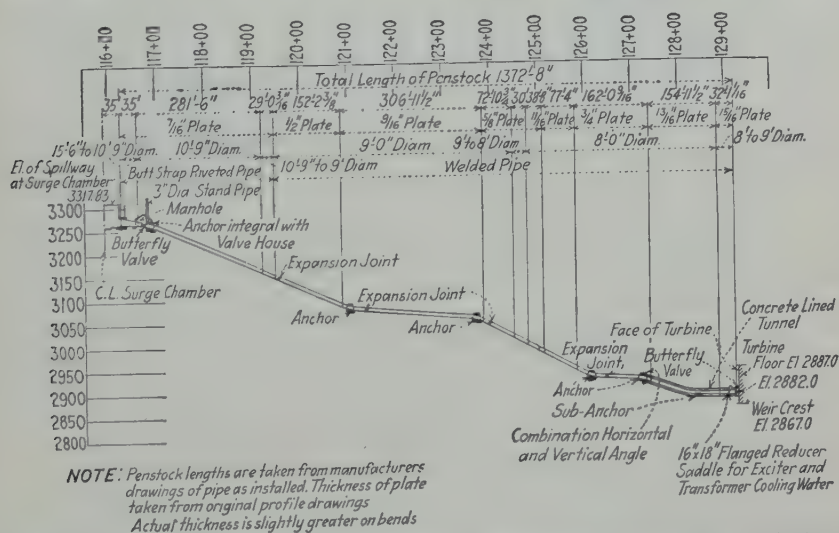


FIG. 149.—Details of penstock layout, Pit River 1 development of Pacific Gas and Electric Company.

the Pit River 1 development of the Pacific Gas and Electric Company are shown in Fig. 149, showing location of expansion joints as well as anchorages. In Fig. 150 details of one of the anchorages are given.

The forces of chief importance acting upon an anchorage at a horizontal bend in the pipe are:

1. That due to the dynamic pressure of the flowing water in changing direction at the bend and acting outward from the bend.
2. That due to the resultant of the static pressure (a) on the upstream end section, and (b) on the downstream end section of the bend and acting outward.
3. If expansion joints are a considerable distance from the bend in each direction, there will be at times a resultant outward pressure at the bend due to temperature stress in the pipe.
4. On a steep slope the weight of the pipe both above and below the anchorage may produce components of outward force at the anchorage sufficient to require consideration. On a long, steep slope, where the pipe

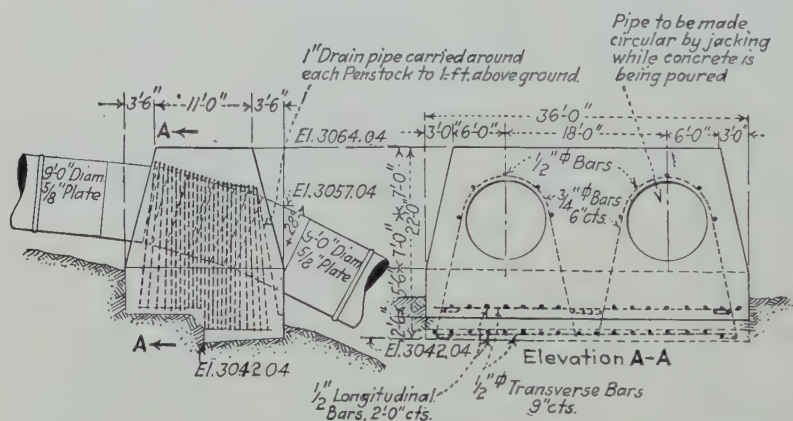


FIG. 150.—Details of anchorage in penstock line, Pit River 1 development.

is straight, anchorages will also be required to hold the pipe weight at occasional intervals.

Where a bend or angle occurs in the grade line of the pipe and the bend is downward, the forces acting are similar to those described but act outward (approximately vertically) from the band. If the bend is upward, the forces tend to hold the pipe down or produce pressure on the pipe support or foundation.

The anchorage at a bend should be designed to carry the forces outlined above, limiting the coefficient of friction in sliding to about 0.5 and keeping the resultant pressure within the base middle third. Depending upon soil conditions, the maximum allowable pressure on the foundation may also have to be considered.

The weight of pipe and water in it provides of itself a resisting force to those forces acting at the bend as outlined previously. Such a resisting force, however, would stress the pipe, particularly in the circular joints, perhaps to an undesirable extent near the bend, and it is better practice to ignore the weight of pipe and water in it in proportioning the anchorages.

The concrete anchorage block should be held together and around the pipe by hoop reinforcement, as well as some longitudinal steel. Steel angles riveted around the pipe are also used to connect pipe and anchorage properly.

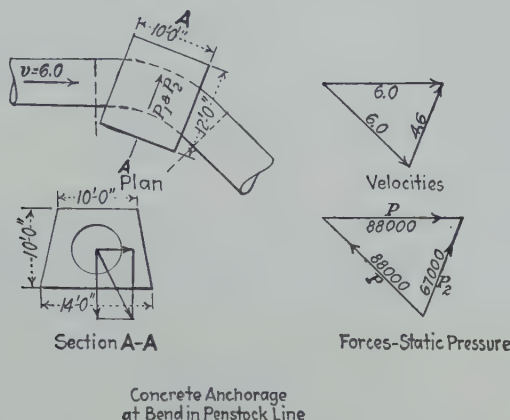


FIG. 151.

In Fig. 151 the analysis of anchorage requirements for the simple case of a horizontal bend is shown. The assumed conditions are as follows:

Diameter of pipe.....	6	ft.
Maximum discharge.....	170	sec.-ft.
Head at bend.....	50	ft.
From which:		
Area of pipe.....	28.2	sq. ft.
Velocity, maximum.....	6.0	ft. per second.

Considering only the first two of the forces listed previously, based upon the graphical determinations of velocity and force triangles shown:

$$P_1 = \frac{170 \times 62.5}{32.2} \times 4.6 = 1520 \text{ lb.}$$

$$P = 28.2 \times 62.5 \times 50 = 88,000 \text{ lb.; whence } P_2 = 67,000 \text{ lb.}$$

$$\text{Total outward force} = P_1 + P_2 = 68,500 \text{ lb.}$$

$$\text{Cubic yards of concrete required} = \frac{68,500}{27 \times 140 \times 0.5} = 36,$$

which will require an anchorage about as shown, bringing the resultant pressure approximately at the outer third point. The average pressure on the foundation will be

$$67/140 = 0.5 \text{ ton per square foot}$$

and the maximum pressure about 1.0 ton per square foot, which is moderate.

The penstock line shown in Fig. 149 should be studied with reference to arrangement of expansion joints and anchorages, and the anchorage at station 123 + 70 shown in Fig. 150 analyzed and checked with reference to the forces acting upon it.

Cradles and Footings.—Cradles or piers with suitable footings will be required for steel penstock lines at intervals of 15 to 25 ft., depending upon thickness and size of pipe. Two conditions must be kept in mind as affecting spacing of cradles or pipe supports, *viz.*:

1. The pipe must not be overstressed as a beam by its loading of water plus its own weight.



FIG. 152.—Thirteen-foot six-inch steel penstock line at Potsdam, N. Y. (Courtesy of Walsh's Holyoke Steam Boiler Works.)

2. Bearing pressures upon the footing foundations must not exceed the safe amount (see page 418 for bearing value of different soils).

For the pipe:

$$\text{The section modulus } S = \frac{\pi(D_2^4 - D_1^4)2}{64 \times D_2} = \frac{\pi(D_2^4 - D_1^4)}{32D_2}$$

Then, if the load per foot of pipe and water is W , for a span l , in feet and allowable bending stress f ,

$$\frac{Wl^2}{12} \times 12 = Sf \text{ and } l = \sqrt{\frac{Sf}{W}} \quad (8)$$

Usually, the proper value of l will be much less than the amount obtained by Eq. (8) in order to comply with the requirements for bearing on the footing foundations, without having to make the footings too wide.

Thus, with an 8-ft. steel pipe $\frac{3}{8}$ in. thick by Eq. (8),

$$l = \sqrt{\frac{2540 \times 7000}{3550}} = 71 \text{ ft.}$$

The weight per foot of pipe and water, however, is about 1.75 tons, which, with concrete cradle footings 18 in. wide at the base and 10 ft. long, will require a spacing of about 21 ft., with a foundation pressure of 2.5 tons per square foot. The spacing of cradles, therefore, should be about 20 ft.

The cradle footings, usually of concrete, should reach suitable bearing material and a depth sufficient to avoid trouble from frost, usually at least 2 ft. below ground level.

The cradles may be of reinforced concrete, or of steel-frame construction as shown in Fig. 152, which shows a 13-ft. 6-in. steel penstock

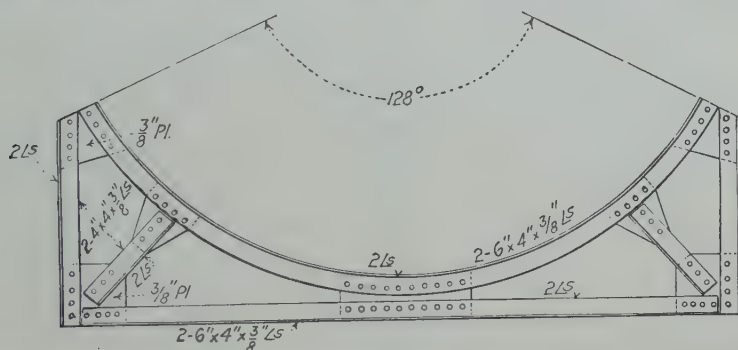


FIG. 153.—Detail of steel saddles for penstock lines. (Courtesy of Walsh's Holyoke Steam Boiler Works.)

5200 ft. long, constructed during 1925 at Potsdam, N. Y., for the St. Lawrence Valley Transmission Company. The steel saddles are spaced 16 ft. on centers. In Fig. 153 details of such steel saddles for pipe sizes from 8 to 14 ft. are given. The steel frames simply rest on the concrete footings and, thus, permit slight movement of pipe to accord with temperature change. Such cradles may be used even on fairly steep slopes, being held in place by the weight of the pipe.

Covering of Steel Penstocks.—Practice, on the whole, favors leaving a steel penstock line exposed. Covering the pipe may be desirable in some cases for the following reasons:

1. Where topography is such that there is danger from slides of snow, rock, or earth.
2. To reduce the number of expansion joints.
3. For small lines, piers will not usually be economical and the pipe may be better supported by covering.
4. To lessen the opportunity for ice to form on the inside of the pipe shell during freezing weather.

The advantages of an exposed line are:

1. Usually cheaper, on account of eliminating most of the excavation, particularly in hard material.
2. Greater accessibility for construction, repairs, painting, and maintenance.
3. Less deterioration and, hence, longer life.

Where the backfill is of porous materials giving good drainage, there is little deterioration of steel pipe if well painted when laid; but, if surface waters are held in contact with pipe by clay or impervious material, rusting and pitting of the pipe shell will take place. In all soils the corrosive action seems to be most rapid on the sides and for a depth of about 12 in. below the surface, due apparently to extreme variations of temperature. Backfilling to the middle of the pipe has no advantage other than in lessened cost.

An alternative to covering the penstock with earth consists in laying 2-in. wooden lagging along the pipe, in rows, a short distance apart circumferentially, and covering with two ply of heavy, tarred paper, held in place by wooden battens. This gives a dead air space next the pipe, is effective, usually less expensive than earth fill, and better for the pipe as regards maintenance.

Surface Protection for Steel Penstocks.—The most efficient method of protecting a steel pipe from corrosion and deterioration during use is by means of a suitable coating material applied by dipping at a proper temperature.

The coating material is a straight-run pitch made from coal tar distilled until the naphtha is removed and with sufficient oil to make a smooth coating, tough and tenacious when cold, and not brittle or with tendency to scale off. It will comply with the following specifications:¹

Melting point of a half-inch cube in water, 100 to 110°F.

Free carbon, insoluble in hot benzol, not less than 12 or more than 22 per cent.

On distillation, not more than 1 per cent shall come up to 170°C. and not more than 15 per cent at 300°C. The specific gravity of the total distillate at 300°C. shall not be more than 1.03. The melting point of the residue from the foregoing distillation shall not be higher than 165°F.

Fresh pitch and oil shall be added to the tank when necessary to keep the mixture at the proper consistency. The oil used for this purpose shall consist of a heavy coal-tar oil which has a specific gravity not less than 1.04 at 60°F. and which shall not lose more than 5 per cent of oil when distilled up to 400°F., and not more than 40 per cent of oil when distilled up to 450°F. The recommended dipping temperature of this specification is 300°F.

The usual procedure is to test a section of pipe and then, while cold, thoroughly clean it both inside and outside from all rust, dirt, and scale, prior to dipping. An electrically operated vertical type of kettle is

¹ As used by Walsh's Holyoke Steam Boiler Works.

used, with heating units so graduated that a uniform temperature can be maintained throughout the whole kettle. By experiment the best dipping temperature has been found to lie within a comparatively narrow range, and great care must be taken to adhere closely to this temperature in dipping. In Fig. 154 an electrically heated dipping kettle is shown with length of pipe, just after dipping.

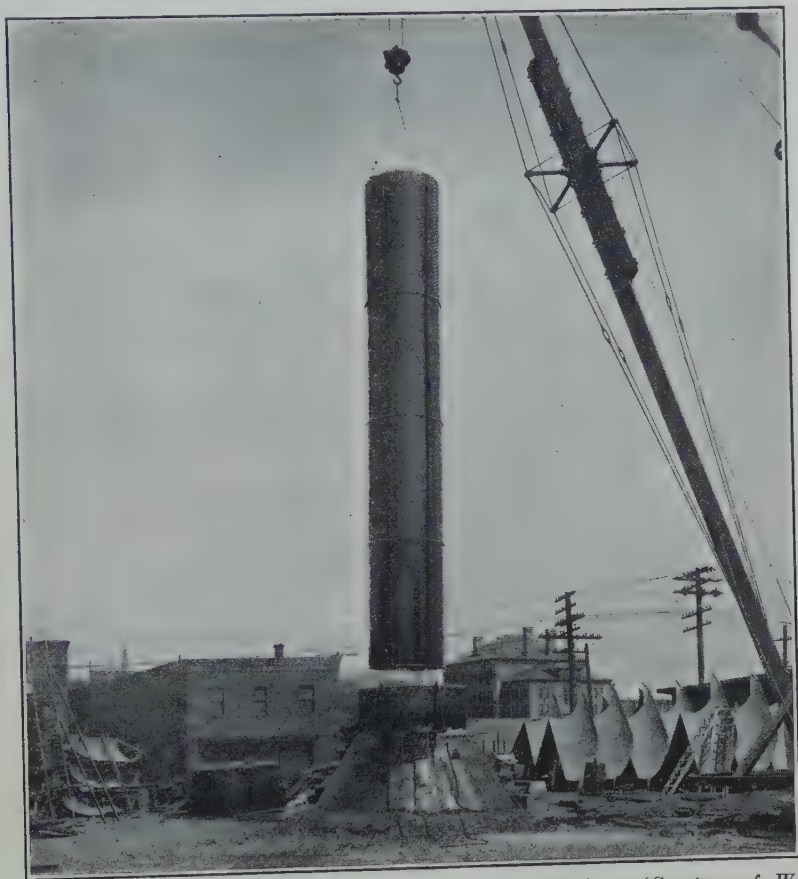


FIG. 154.—Electrically operated dipping kettle for steel pipe. (Courtesy of Walsh's Holyoke Steam Boiler Works.)

Where painting is used for protecting steel pipe, for the shop coat linseed oil is sometimes used, or some clear primer that will not cover defects in pipe and yet will protect it against rust during shipment. For the field coat graphite, red lead, and asphaltum paints have all been used, the best practice being to paint both inside and outside of pipe. If red lead is used as a first coat, it must be immediately protected by a second coat of graphite, asphalt, or similar paint.

REINFORCED-CONCRETE PENSTOCKS

The use of reinforced concrete for penstocks is now quite common, limited in general, however, to heads less than about 60 ft., this limitation applying particularly in colder climates, where alternate freezing and thawing tend to cause concrete under high water pressure to deteriorate rapidly.

For moderate pressures, reinforced concrete may be more economical than steel-riveted or welded pipe, as these latter must usually be of some minimum thickness for practical reasons and, hence, are wasteful of metal. The steel reinforcement of the concrete pipe, however, can be exactly proportioned to fit the given conditions of pressure.

The size and spacing of circumferential reinforcement bars are determined from the fundamental hoop-tension formula. For illustration, if the diameter d of a circular penstock is 8 ft. and it is to be subjected to a pressure head of 50 ft., the stress per linear inch carried by the bars will be

$$T = \frac{pd}{2} = 50 \times 0.434 \times 48 = 1050 \text{ lb.}$$

If $\frac{3}{4}$ -in. square twisted bars are used at 16,000 lb. per square inch, their spacing will be

$$\frac{9}{16} \times \frac{16,000}{1,050} = 8.5 \text{ in. center to center}$$

The longitudinal bars to take care of temperature and shrinkage stresses should be from 0.25 to 0.50 per cent of the cross-sectional area of the concrete, depending on conditions of temperature range and whether or not the penstock is covered.

The thickness of concrete is fixed by practical considerations, being proportional to the size of penstock and of sufficient thickness to get a relatively impervious shell of good concrete, which should be in the proportion of about 1:2:4 or richer. For best results, proportions approximating the "ideal mix" should be determined for the particular aggregates used.¹ A suitable thickness of concrete will be obtained if the thickness of the pipe shell in inches is made equal to the diameter of the pipe in feet, with a minimum thickness of shell, however, of 8 in.

In the above example, assuming a thickness of concrete of 8 in., the area of longitudinal reinforcement would ordinarily be about

$$0.003 \times 0.785 (9.33^2 - 8.0^2) \times 144 = 7.85 \text{ sq. in.,}$$

or, say, about 30 $\frac{1}{2}$ -in. bars spaced approximately 11 in. apart.

The invert of the concrete penstock should be somewhat thicker than the shell. A good arrangement is shown in Fig. 155, as used for the

¹ See TAYLOR and THOMPSON, "Concrete, Plain and Reinforced," Chap. X, 1922.

concrete penstock of Plant 5 of the New England Power Company, assuming that, normally, the penstock will be built as far as possible on firm ground. If it becomes necessary to construct on an earth fill, the bottom (exterior) of the penstock should be horizontal of width equal

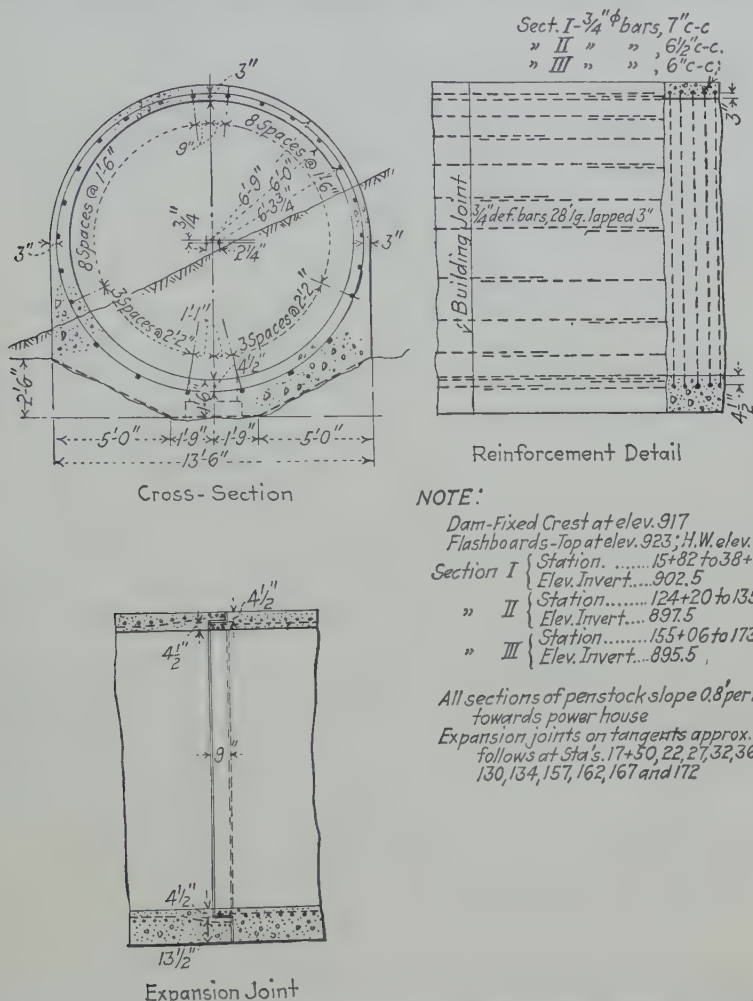


FIG. 155.—Twelve-foot concrete penstock details, Plant 5, New England Power Company.

to the external diameter, so that the exterior side walls, up to the level of the center of the penstock, may be vertical. In such a case the earth fill should be placed in relatively thin layers, not over 5 or 6 in. and thoroughly rolled, with, preferably, some time for settlement before constructing the penstock.

It will usually be worth while to cover a concrete penstock with 2- or 3-ft. depth of fill on top, to minimize expansion and contraction due to

NOTE:

- Dam-Fixed Crest at elev. 917
Flashboards-Top at elev. 923; H.W. elev. 932
- | | |
|-----------|----------------------------------|
| Section I | { Station. 15+82 to 38+29 |
| | { Elev. Invert. 902.5 |
| " II | { Station. 124+20 to 135+40 |
| | { Elev. Invert. 897.5 |
| " III | { Station. 155+06 to 173+00 |
| | { Elev. Invert. 895.5 |

All sections of penstock slope 0.8' per 1000 towards power house
Expansion joints on tangents approx. as follows at Sta's. 17+30, 22, 27, 32, 36, 125, 130, 134, 157, 162, 167 and 172

temperature changes and tendency to leakage. This was done in the case of the penstock shown in Fig. 155.

The use of contraction joints may be desirable in order to prevent random cracks occurring. One form of such a joint is shown in Fig. 155, where, as will be noted, the reinforcement bars must be broken. The joint itself is filled with an elastic tar pitch or asphalt paint. Such joints generally tend to tighten under use, due to the action of sediment in the water. As a matter of fact, several concrete penstocks have been built with no contraction joints, which have cracked more or less but tightened under use. It is desirable, where possible, to construct the penstock in cool weather, so that most of the time it will be under some compression due to temperature expansion.

WOOD-STAVE PENSTOCKS

Wood-stave penstocks are now in very general use in all parts of the country, particularly for moderate heads. Wood-stave construction, like concrete, is best adapted for the portions of a penstock line where, due to the minimum thickness requirement, steel pipe is not economical, or, say, up to about 150-ft. head. For heads above about 150 ft., substantially the same amount of steel will be required (assuming like values of tensile strength), whether the steel is used in plate form or as bands, modified somewhat, however, by the fact that the bands are or can be 100 per cent efficient, while the plates are limited by joint efficiency to 70 or 80 per cent. The net result is an ordinary limitation in the use of wood-stave pipe to heads under about 150 ft.

Design of Wood-stave Penstock.—The design of a wood-stave penstock is consistent with the fundamental hoop-tension formula, modified to suit the special qualities and limitations of the wood staves.

Assuming $S = \pi r^2 f$, where S is strength of (circular) band in pounds, r is radius of band in inches, and f is allowable tensile strength of band in pounds per square inch, and, if the wooden stave of thickness t in. can safely carry e lb. per square inch in bearing across the grain, then

For maximum size of bands:

$$S = \pi r^2 f = (R + t)(e \cdot r \cdot 1) \quad (9)$$

assuming the band to have an effective width in bearing $= r$. This value of S must not be exceeded, or the bearing stress of band or stave will exceed the safe value of e , the latter usually being taken at 650 lb. per square inch.

For spacing of bands:

Using the value of S from Eq. (9), or for any size of band with lesser value of S , the distance between bands will be

$$d = \frac{S}{pR + e't} \quad (10)$$

where e' is the crushing strength of the staves along the grain, due to swelling of the wood after the bands are tightened, e' usually being about 100 lb. per square inch.

Using the maximum size of band will often result in a value of d too large for practical use. The latter should not exceed 8 to 10 in., or the pipe is likely to leak.

Numerical example:

8½-ft. wood-stave pipe, 2-in. staves, $f = 15,000$ lb. per square inch, head = 50 ft.

For maximum size of bands:

$$\begin{aligned}\pi r^2 \times 15,000 &= (51 + 2.5) 650r \\ r &= \frac{53.5 \times 650}{\pi \times 15,000} = 0.74 \text{ in.} \\ \therefore S &= \pi \times 0.74^2 \times 15,000 = 25,800 \text{ lb.}\end{aligned}$$

For spacing of bands:

$$d = \frac{25,800}{50 \times 0.43 \times 51 + 100 \times 2.5} = \frac{25,800}{1100 + 250} = 19.2 \text{ in.}$$

which is too great to use.

If d is taken at 10 in.,

$$\begin{aligned}10 &= \frac{S}{1090 + 250}; S = 13,400 = \pi r^2 \times 15,000 \\ r^2 &= 0.284; r = 0.53 \text{ in.}\end{aligned}$$

or approximately 1-in. bands spaced 10 in. center to center.

It will usually be possible, on a penstock line, to adopt a given size of band and vary the spacing with the head.

The staves will vary somewhat in thickness with the size of line. Horton¹ has derived an empirical formula for stave thickness as follows:

$$T = 1 + \frac{h}{100} + \frac{d}{100}, \quad (11)$$

where

T = thickness of stave, inches.

h = head, feet.

d = pipe diameter, inches.

Practical Details (see Fig. 156).—Staves are usually of redwood or fir milled on the edges to true radial planes, fitted with tongue and groove, and milled on the outside and inside faces to the correct curvature for the pipe size. The staves are trimmed square at the ends to form the butt joints, which break joints at least 24 in. The ends of the staves have a saw kerf cut across the face for the insertion of the metal tongues. The width of the kerf is exactly the thickness of the tongue; the depth of

¹ *Eng. Record*, Mar. 20, 1915, p. 356.

the kerf is slightly less than half the width of tongue; and the length of the tongue is slightly in excess of the finished width of the stave, measuring along the saw kerf, so that, in the finished pipe, a water-tight joint is made between the butt ends of the two staves and the sides of the adjacent staves.

The tongues are made from band steel and are usually about $\frac{1}{8}$ by $1\frac{1}{2}$ in., of a length depending on the width of stave.

The bands, usually $\frac{3}{8}$ to 1 in. in diameter, consist of steel rods, circular in cross section, with a head at one end and a thread at the other end, the two ends being united by a malleable iron shoe (Allen type), the rod being tightened with a nut and washer. For pipes of larger diameters the rods

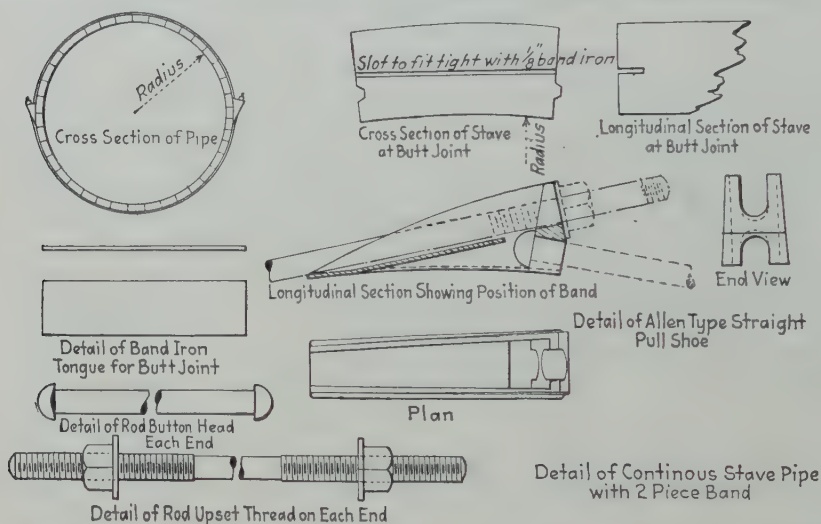


FIG. 156.—Details of wood-stave pipe construction. (Redwood Manufacturers Company.)

are in two sections, one section being double headed and the other section being double threaded. In this case two malleable iron shoes are required for a complete band. In pipe of 10 ft. diameter or over, three rods and three shoes would form a complete band. The threads on the rod are cold rolled on upset ends, and the dimensions of the head and thread are such that the thread and head ends are as strong as the body of the rod. The bands have a tensile strength of 55,000 to 65,000 lb. per square inch and are furnished according to standard steel specifications. The Allen-type shoe is so shaped that it fits closely upon the outside of the pipe and the cinching of the rod produces a straight pull, the entire bands, when in place, lying in a plane perpendicular to the direction of the pipe. The shoe is so designed as to develop the full tensile strength of the band. If shoe and band were tested to destruction, the band would break first, which enables the factor of safety to be accurately determined by the tensile strength of the band.

In erection (see Fig. 157) the staves are assembled in the trench and put together to form a circle of the diameter of the pipe, and the bands put around the outside and tightened to hold the staves together. End joints in staves are broken by a lap of at least 2 ft. and the pipe put together continuously in the trench, the metal tongues being inserted as previously described.

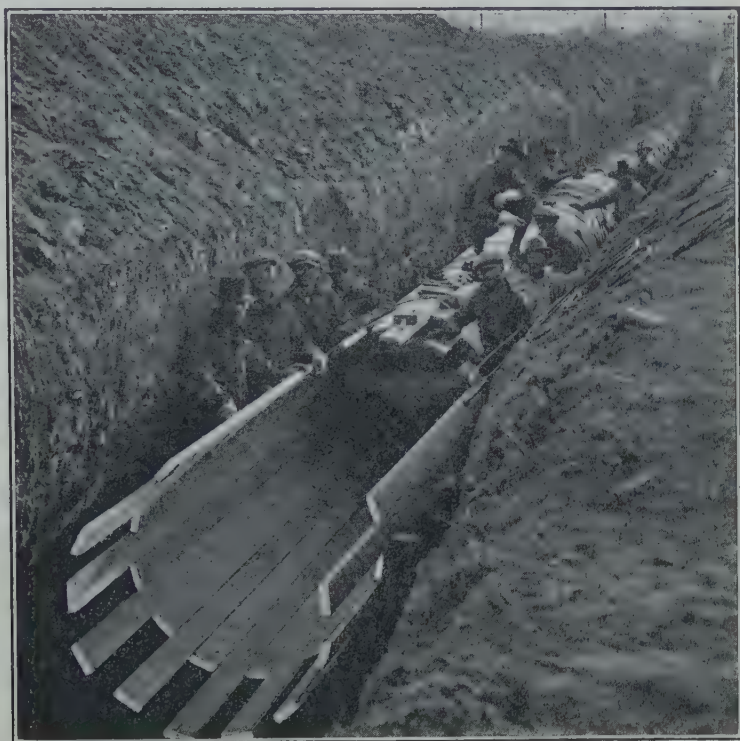


FIG. 157.—Erection of wood-stave pipe line. (Courtesy of Redwood Manufacturers Company.)

Fairly sharp curves can be obtained by bending the staves, the approximate limit in sharpness of curve being as follows:

Diameter of Pipe, Feet	Shortest Radius, Feet
4	125-150
5	150-175
6	200-225
7	275-300
8	305-400
9	450-500
10	600
11	650
12	700

After the pipe is completed and before the water is turned in, the bands are gone over and tightened uniformly so as to place an even tension on them. When the pipe is filled with water, the staves swell sufficiently to bed the bands slightly into the wood and make longitudinal joints water-tight.

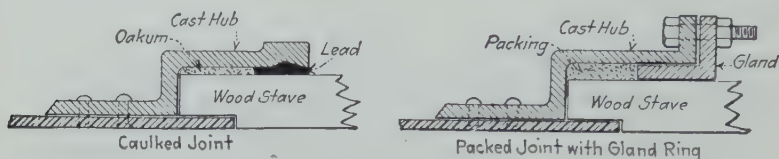


FIG. 158.—Methods of connecting steel and wood-stave pipes.

Sharp bends in wood-stave penstocks are usually made by the use of short steel-pipe sections curved to the requisite amount. The connection between wood-stave and steel pipe may be made by either of the two methods shown in Fig. 158, one of these using a calked joint of lead and oakum, as commonly employed for cast-iron pipe, and in the other a packed joint with gland ring.

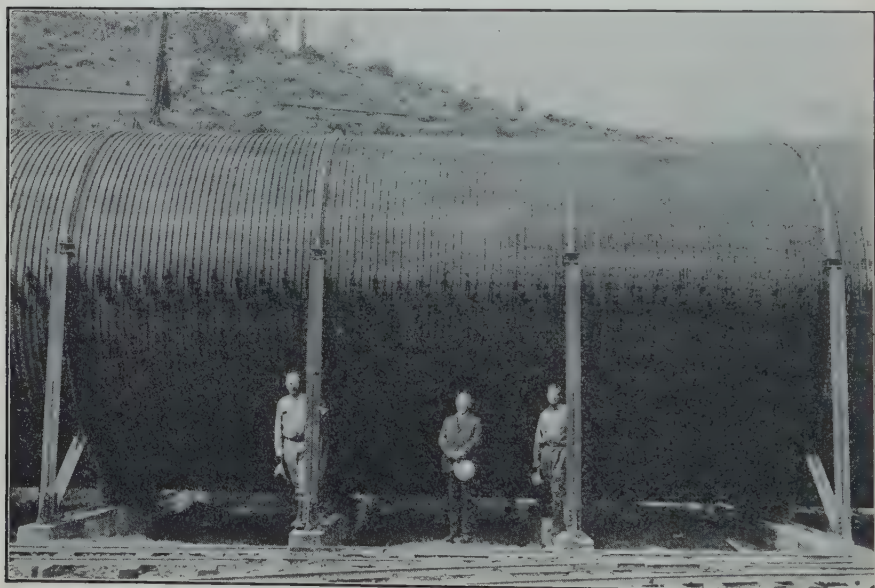


FIG. 159.—Sixteen-foot wood-stave penstock at Copco Plant 2 of California-Oregon Power Company. (Courtesy of Federal Pipe and Tank Company.)

Cradles, usually of concrete, are spaced 8 to 12 ft. apart on centers. In the case of the Searsburg plant of the New England Power Company, built in 1921, reinforced-concrete precast cradles were used, set on concrete sills.

passing over the top of the pipe, with ends bolted to the top of each cradle, takes the place of a band at the cradle section. The details of bands, cradles, and footings in Fig. 159 should be studied.

Wooden cradles are also in very general use for wood-stave penstocks. In Fig. 160 the arrangement for smaller sizes of pipe is shown, and in Fig. 161 the wooden cradles are shown as used for the 13.5-ft. wood-stave penstock line of the White Salmon (Condit) plant, near Portland, Ore., serving two 9000-hp. wheel units under a head of 160 ft. at 360 r.p.m. There are 1120 cradles in a total length of penstock of 5100 ft.

ACCESSORIES—PENSTOCK LINES

Air Vent.—An air vent of ample size should always be installed just below the gates at the upper end of the penstock, to permit air to enter when the head gates are shut and water drawn out of the penstock.

The maximum air requirement, with head gates closed and wheel gates wide open, would evidently be the same number of cubic feet of air per second as the discharge; but, as the penstock water level lowered, the head on the wheels would lessen and the wheel discharge would diminish, varying as the square root of the head on the wheel. Furthermore, in draining the penstock the wheels would, with a careful operator, be at only part gate. It is safer, however, to provide the air vent for possible full discharge.

Thus, with an 8-ft. penstock and discharge of 300 sec.-ft., as for the Searsburg plant (Table 69, page 360), 300 cu. ft. of air per second may have to enter the air vent for a time. A 3-ft. vent pipe is provided at the head gate house of this plant which would give a possible air velocity through the vent pipe of about 43 ft. per second to provide air enough to keep the pipe filled with air as the water was drawn off. While this air requirement is unlikely with a careful operator, it is evident that the air vent should be ample, as in this case. It is also essential that the gate house be so arranged that air can enter freely at all times.

In one case, at a plant in Connecticut some years ago, the water in the vent pipe became frozen over, resulting in collapse of the penstock line, when the head gates were shut and water drawn out. It is, therefore, also essential that the air vent be protected from frost so as surely to function at all times. A simple means of effecting this is by the use of a small electric heater floating on the water in the top of the vent pipe.

Surge Tank.—Usually a surge tank is required in the penstock line, located as near as practicable to the power house for purposes of speed and pressure regulation. The principles involved and arrangement of the surge tank are discussed in Chap. X.

Air Valves.—An air valve to permit air to enter the pipe may be required at the lower end of a flat grade where the penstock line begins

to pitch suddenly to the power house. In case of sudden load acceptance by the wheels, the water may be drawn out of the lower, steep portion of the penstock faster than it can flow in the flatter part, thus tending to cause a collapsing pressure on the pipe at the change in grade. The requirements for and arrangement of air valves are also discussed in Chap. X.

RELATIVE CAPACITY OF DIFFERENT PENSTOCKS

The various kinds of penstocks described—steel, concrete, and wood stave—differ considerably in their carrying capacity, due to difference in frictional effect.

Based on Table 63, King's "Hydraulic Handbook," values of n in the Manning formula are as follows:

Kind of pipe	n	Inverse ratio to n for wood pipe = 1.00
Clean cast iron.....	0.012	0.92
Clean riveted steel.....	0.015	0.74
Lock-bar steel.....	0.011	1.00
Clean wood.....	0.011	1.00
Concrete.....	0.013	0.85

Keeping in mind that, for a given hydraulic radius, capacity varies practically inversely as n , the relative hydraulic advantage of wood-stave, lock-bar, and concrete over riveted-steel pipe is obvious from the above table.

The different types of steel pipe also show considerable variation in friction losses. A summary of tests by the Pacific Gas and Electric Company¹ gives the following values of n in the Kutter formula:

Kind of steel pipe	n	Inverse ratio to $n = 0.015$
Lap-welded, bump joints.....	0.013	1.15
Thin pipe, lap joints.....	0.014	1.07
Moderately thick pipe, butt joints.....	0.016	0.94
Heavy pipe, triple-riveted butt joints.....	0.018	0.83

As will be noted, in general, joints of high-strength efficiency are lower in hydraulic efficiency, with the exception of lock-bar steel pipe, which

¹ Report of Hydraulic Power Committee, Nat. Electric Light Assoc., 1923, p. 8.

is high in both. The bump joint is apparently most efficient hydraulically for riveted-steel pipe.

PENSTOCK CONSTRUCTION COSTS

Riveted-steel Pipe.—Based upon a 200 per cent cost-index factor (see Table 122, Chap. XII), the steel for straight pipe will cost from 4 to $4\frac{1}{2}$ cts. per pound. In estimating weight of pipe based upon the pipe thickness, about 10 per cent should be added to allow for lap of plates, butt plates, rivets, etc. The cost of pipe erected, but not including cost of unloading and hauling to site, will be from $5\frac{1}{2}$ to 8 cts. per pound, depending chiefly upon the difficulties of erection. Unloading and hauling will cost generally from $1\frac{1}{2}$ to 2 cts. per pound, so that the total cost of pipe erected is likely to be from 7 to 10 cts. per pound. Additional items of cost will be:

1. Excavation and grading, costing from \$1 per cubic yard upwards, depending on conditions.
2. Cradles, which may be of steel and included at about the same price per pound as the pipe or of reinforced concrete, with suitable unit costs.
3. Footings, usually of concrete.

Wood-stave Pipe.—The requirements for bands and staves, and estimated cost of redwood pipe erected, including a freight allowance of 90 cts. per 100 lb. (or, practically, the freight from coast to coast), but not

TABLE 71.—WOOD-STAVE PIPE REQUIREMENTS AND ESTIMATED COSTS

Diameter, feet	Head, feet	Band diameter, inches	Band spac- ing on centers, inches	Finished stave thickness, inches	Estimated cost, dollars per running foot erected
4	30	$\frac{1}{2}$	9.47	$1\frac{1}{2}$	5.30
	50	$\frac{1}{2}$	5.04	$1\frac{1}{2}$	5.93
	150	$\frac{5}{8}$	1.60	2	9.78
6	30	$\frac{5}{8}$	9.08	$2\frac{1}{2}$	12.77
	50	$\frac{5}{8}$	5.08	$2\frac{1}{2}$	14.19
	150	$\frac{3}{4}$	1.29	$2\frac{1}{2}$	23.00
8	30	$\frac{3}{4}$	10.00	$3\frac{1}{2}$	21.68
	50	$\frac{3}{4}$	6.38	$3\frac{1}{2}$	23.58
	80	$\frac{3}{4}$	3.09	$3\frac{1}{2}$	27.44
10	30	$\frac{3}{4}$	8.05	$3\frac{1}{2}$	29.88
	50	$\frac{3}{4}$	5.10	$3\frac{1}{2}$	32.12
	80	$\frac{3}{4}$	3.02	$3\frac{1}{2}$	36.10
12	30	$\frac{3}{4}$	7.08	$3\frac{1}{2}$	33.24
	50	$\frac{3}{4}$	4.25	$3\frac{1}{2}$	38.16
	80	$\frac{3}{4}$	2.65	$3\frac{1}{2}$	46.50

NOTE: Maximum band spacing = 10 in.

including cost of grading, cradles, footings, or unloading and hauling pipe, are given in Table 71¹ for a considerable range in size of pipe and head. These costs are as of 1926 or, approximately for a 200 per cent cost-index number.

Redwood cradles similar to those shown in Fig. 160, page 381, spaced 12 ft. on centers on tangents, cost about 85 cts. per linear foot for 4-ft. pipe and \$1.25 per linear foot for 6-ft. pipe. Above 6 ft. in diameter, concrete cradles are usually cheaper.

¹ Furnished by Allen V. Garratt, eastern states representative of Redwood Manufacturers Company.

CHAPTER VII

POWER HOUSE AND EQUIPMENT—HYDRAULIC AND STRUCTURAL

GENERAL ARRANGEMENT

The power house of a hydroelectric development should be regarded as a protective covering for the equipment, and as such, its size and shape will vary greatly with its location and the use to be made of the power generated. The generators, switchboard, and low-tension switching equipment should always be covered or protected from weather. It is becoming more and more the practice to place the transformers and high-tension switching equipment out of doors near the power house, which results in a material saving in cost.

With the foregoing ideas in mind, the general layout of the hydroelectric units, with their auxiliaries, should be made first, in the best manner; then the power house should be planned to include them as well as the necessary space for switchboard, low-tension switches, bus bars, repair room, shop, office, etc.

As the power house is the conspicuous and vital part of an hydroelectric development, it should be of satisfactory appearance, harmonizing with the surroundings. A simple and dignified design will give the best effect and can usually be obtained at reasonable cost. Both concrete and brick (of various kinds) are in general use for the superstructure, and concrete for the foundations and substructure. Good examples of power-house design as regards appearance are shown in Figs. 179 to 181.

EQUIPMENT TO BE PROVIDED FOR

Following is a list of the more important items of equipment to be provided for in the layout of the power house:

Hydraulic equipment:

- Turbines.

- Gates or gate valves, and hoists or controls.

- Racks for low heads, and rack-raking device.

- Relief valves for penstock settings.

- Governors:

 - Oil pumps.

 - Pressure system.

- Flow-measurement equipment:

 - Gages, head and tailwater levels.

 - Provision for discharge measurements.

Electrical equipment:

Generators:

Air ducts.

Exciters, direct, belted or motor operated.

Transformers:

Pumps and cooling system.

Connections.

Runs and platforms.

Switching equipment:

Low-tension buses.

Switchboard panels.

Switchboard equipment and instruments.

Oil switches.

Reactors.

High-tension system:

Buses.

Oil circuit-breakers.

Lightning arresters.

Outgoing connections.

Auxiliaries:

Storage batteries.

Station lighting.

Miscellaneous:

Crane.

Shop.

Lavatory.

Office.

GENERAL ARRANGEMENT OF HYDROELECTRIC UNITS

Reaction-wheel Plants.—With the prevailing type of vertical single-runner water-wheel units and direct-connected generators, the most convenient and economical arrangement is that with the units in a line parallel to the length of the power house. The spacing of the units will usually be fixed by the necessary width of flume or scroll case at entrance to the wheel or at the head gates, or draft tube at its mouth or occasionally by the overall diameter of the generators. In this arrangement the flumes or penstocks will usually approach the power house in a direction perpendicular to its longer axis, although with penstock settings the approaching lines may be sometimes inclined to the power-house axis, due to limitations of location.

With horizontal units the best arrangement is usually to place each unit on a line normal to the axis of length of the power house. The spacing, as with vertical units, will be controlled by flume, draft tube, tailrace, or generator dimensions. This arrangement would always be used with horizontal units and open wheel-pit construction, *i.e.*, for low heads. For high heads, double-runner developments have been made, with the axis of each unit parallel to the longer axis of the station, as in the White River plant near Seattle, Wash. (1912), and the White

Salmon River plant near Portland, Ore. This, however, requires a long building, and undoubtedly such developments would now be made with vertical units.

Impulse-wheel Plants.—For the typical horizontal setting of impulse wheels, the ordinary arrangement is to place the wheel shaft parallel to the long axis of the power house, the turbine and generator being direct connected. The penstock would thus approach in a line normal to the axis of length of the house, and water would leave the wheel buckets in this same general direction, which would also be that of the tailrace channel. The spacing of units in this case is determined by their dimensions and necessary clearance, rather than by penstock, flume, or tailrace widths, as the discharge of wheels of this type is not usually large.

Vertical settings of impulse wheels with multiple nozzles are not common, but, if used, the units should lie with centers on a line parallel to the axis of length of the house, both penstock and tailrace channel running in this same direction.

General Features of Power House.—The typical and common arrangement of units in a line parallel to the axis of length of the power house results in a rectangular building with main floor occupied by the wheel-generator units and their accessories, the governors, oil pumps, exciters, etc. and, frequently, in moderate-sized stations, the switchboard and low-tension buses, and switches. The switchboard and operating platform are frequently placed on a mezzanine or second floor.

In a two- or three-story building, the transformers and oil switches may be on the second floor, although the former are more commonly placed on the first-floor level in a line parallel to the length of the station. It is becoming more and more the practice to separate the phases in switching, which, with a vertical arrangement of cells, results in a four-story building—one for each phase and an upper one for operating.

More often the main station has one floor (and perhaps a basement just above wheel level) and sufficient height for clearance and use of the crane in handling machinery. A bay of two or more stories at the rear of the main floor is commonly used for low-tension buses, transformers, oil switches, and high-tension connections and auxiliaries. Often the first floor of this bay, due to a sloping ground location, is at a higher elevation than the floor of the main house.

In the outdoor station the transformers and high-tension equipment and connections are placed in a yard outside and near the power house. For large stations a rear bay will commonly be required for low-tension buses, switches, etc., but this may not be necessary for smaller stations with outdoor yards.

Arrangement of Units.—Types of wheel settings have been discussed in Chap. IV. Following is an outline classification (the common arrangement *italicized*):

A. Reaction wheels:

1. *Vertical* or horizontal shafts.
2. *Single* or multiple runners (from two to six or more have been used).
3. *Open wheel-pit, concrete spiral-flume* or *metal scroll-flume* settings. These are used progressively in the order given as the head increases, the first two commonly for concentrated falls or canals and the last for penstocks.

Cylindrical steel casings have been used for horizontal settings, with single or multiple runners.

B. Impulse Wheels:

1. *Horizontal shafts (single or double overhung)* or vertical shafts.
2. *Single* or multiple nozzles (two to six).

SELECTION OF WATER-WHEEL UNITS

As stated in Chap. III, the basis for determination of wheel capacity is a flow-duration curve at the power site. The wheel capacity to be installed is, theoretically, a problem of economics—that capacity being best which shows the greatest net return in value of power produced. The problem is not a simple one, however, as an increase in wheel capacity will add not only to wheel cost but to the cost of generators as well and perhaps, to some extent, of power-house foundations, although, as further explained (page 664), the “increment cost” of added capacity is much less per horse power than the cost per horse power of the entire development, including dam and waterway. Then, too, the value of the power produced and the load factor may not be known exactly. Where these latter factors are fairly well defined, however, as in a new plant in a power system that has been in use for some time, it may be worth while to estimate carefully the cost of power for several different wheel installations and use the resulting figures as a guide in deciding what capacity of wheels to install.

In the consideration of an isolated plant, the problem is further complicated by the necessity of allowing for a spare unit, which will be discussed later.

In Table 72 are given data of actual wheel installations which are useful as a general guide in this matter. These capacities as used are the resultant of the various factors affecting capacity, *viz.*, installation cost, value of power, character of load, load factor, etc., as well as allowance for spare units, and naturally vary somewhat in different sections of the country. Nevertheless in the modern plant, which is a part of a power system, it will usually be found that with full development a wheel capacity corresponding to a flow available from about 20 to 40 per cent of the time is warranted, depending upon circumstances.

Number and Size of Units.—When the total wheel capacity has been determined or assumed, the next step is to fix the number of units. For a given total plant capacity, the cost of power house and its foundations is directly proportional to the number of units. Moreover, the cost of

TABLE 72.—INSTALLED WHEEL CAPACITIES AT HYDROELECTRIC PLANTS

Num- ber (1)	Station (2)	River and state (3)	Date (4)	Drainage area, square miles (5)	Head, feet (6)	Wheel capacity			Per cent of time dis- charge avail- able (11)	
						Num- ber of units (7)	Total horse power (8)	Discharge Second- feet per square mile (10)		
										Second- feet (9)
1	Rumford Falls.....	Androscoggin, Maine	1918	2,090	100	4	30,000	3,150	1.50	28
2	Great Falls.....	Presumpscot, Maine	1901	438	33	4	3,500	1,150	2.64	25
3	Garvins Falls.....	Merrimack, New Hampshire	1924	2,340	30	3	9,200	3,300	1.41	35
4	Vernon.....	Connecticut, Vermont	1910-1925	6,300	34	10	48,000	15,500	2.46	21
5	New England Power Company 2, 3, 4, 5.....									
6	Searsburg.....	Deerfield, Massachusetts	1912-1915	250-500	66-230	3-4	52,800	300	3.80	21
7	Davis Bridge (Harriman).....	Deerfield, Vermont	1922	98	230	1	6,200	300	3.07	28
8	Sherman Island, present.....	Hudson, New York	1924-1925	184	350-390	3	60,000	1,770	9.60	5±
9	Ultimate.....		1924	2,782	66	4	40,000	6,280	2.26	29
10	Colton, present.....	Raquette, New York	1918	981	258	5	50,000	7,850	2.82	21
11	Ultimate.....					2	22,000	880	0.90	60
12	Bartlett's Ferry, present.....	Chattahoochee, Georgia	1926	4,200	112	2	44,000	1,320	1.35	50
13	Ultimate.....					2	44,000	4,080	0.97	53
14	Tallulah Falls.....	Tallah, Georgia	1914	190	580	4	88,000	8,160	1.94	22
15	Muscle Shoals, present.....	Tennessee, Alabama	1925	30,800	92	5	85,000	1,600	8.4	5±
16	Ultimate.....					8	260,000	28,000	0.95	55
17	Keokuk.....	Mississippi, Iowa	1912	119,000	32	15	150,000	68,000	2.21	25
18	Great Falls (Volta).....	Missouri, Montana	1916	23,000	52	6	86,000	47,000	0.39	50
19	Holter.....	Missouri, Montana	1917	20,000	100	4	58,000	6,000	0.26	75
20	Kerkhoff.....	San Joaquin, California	1921	1,488	315	3	45,000	6,000	0.30	70
21	Oak Grove, present.....	Clackamas, Oregon	1924	266	849	1	35,000	1,530	1.03	100
22	Ultimate.....					3	105,000	1,420	1.58	33
23	White River, present.....	White, Washington	1911-1918	424	480	3	67,500	1,550	3.66	50
24	Ultimate.....					4	90,000	2,060	4.85	25
25	Conditt.....	White Salmon, Washington	1913	350	175	4	90,000	2,060	4.85	25
26	Pitt 3.....	Pitt, California	1925	4,320	280	2	20,000	1,200	3.44	30±
27	Molly's Falls.....	Winooski, Vermont	1926	25	350	3	99,000	3,600	0.83	16±
28	Sherman.....	Deerfield, Massachusetts	1926	232	78	1	7,000	7,207	12.0	3±
29	Fifteen Mile Falls (Comerford).....	Connecticut, New Hamp- shire-Vermont	1930	1,600	170	4	10,000	1,400	6.0	10±
30	McIndoes Falls.....	Connecticut, New Hamp- shire-Vermont	1931	2,200	27	4	215,000	12,000	7.60	2±
31	Bellevue Falls.....	Connecticut, New Hamp- shire-Vermont	1928	5,300	60	3	16,500	5,000	2.27	20±
32	Conowingo.....	Susquehanna, Maryland	1928	27,000	89	7	378,000	45,000	1.67	30±
33	Safe Harbor.....	Susquehanna, Pennsylvania	1931	26,000	53	6	255,000	51,000	1.95	25±
34	Cobble Mountain.....	Little, Massachusetts	1932	48	430	3	33,000	1,040	21.6	0±
35	Caribou, present.....	Feather, California	Impulse Wheels 1921	504	1,008	3	90,000	1,000	2.00	20
36	Ultimate.....					6	180,000	2,000	4.00	30±
37	Big Creek 1, present.....	Big Creek, California			2,050±	2	40,000	215	2.69	0±
38	Ultimate.....					4	80,000			
39	Mystic Lake.....	W. Rosebud, Montana	1924	47	1,050	2	15,000	150	3.20	30±

equipment—wheels, generators, and accessories—will increase with the number of units but not in direct proportion to this. It is, therefore, fundamentally desirable to have as few units as possible, consistent with safe and economical operation. The plant location and use, whether as a part of an interconnected power system or as an isolated interdependent power unit, are, therefore, important.

Isolated Plants.—Considering first an isolated plant, it will be necessary to plan for possible breakdowns and consequent repairs by having a spare unit. Hence, the effective capacity of such a station is the maximum output of all the units but one, at full gate. Usually, in such stations either three or four units will be most satisfactory. With three units each one should be able to carry, at full gate, 50 per cent of the maximum load so that, when necessary, two units may handle the maximum load. With three units this load may then normally be handled with the wheels all at or near the full-load (most efficient) point. With four units in an isolated plant, in similar fashion, each, at full gate, should be able to handle about one-third of the total maximum load.

Partial Installations.—Where the load or demand for power is growing but not fully developed, it may be desirable to lay out the power house and construct foundations and tailrace for the ultimate number of units expected, but to install only a part of these at first. Sometimes in large plants, only a part of the power house (and perhaps penstock feeders) may be built until such time as power demands require greater capacity. Of the plants listed in Table 72:

1. At Davis Bridge two units were installed in 1924 with power house, tailrace, etc., constructed for a third unit, but its penstock feeder was not built. The third unit with its penstock line was installed in 1925.

2. At Sherman Island space was left in the power house for an additional unit (four were installed), but all other construction was completed.

3. At Keokuk 15 units were installed and power-house foundations, only, built for 15 more.

4. The Oak Grove plant now has two units, with a proposed ultimate installation of three units.

5. At the Caribou plant the power house was constructed for three units, and two were installed in 1921, and the third in 1924. Another section of power house for three units, with penstock feeders, will eventually be constructed.

6. The White River plant, built in 1912 with two units, had another unit with its feeder penstock added in 1918, and space in the power house is provided for a future fourth unit.

7. Big Creek 1 has an initial installation of two units, with two more planned.

8. Conowingo had an initial installation of seven units with power-house foundations for four more units.

9. Safe Harbor similarly had six units installed with provision for six more.

Interconnected Plants.—Where a plant is to be simply one of a system, as is often the case, provision for a spare unit is of less importance. Furthermore, steadier operating conditions may often be obtained and also a better load factor, so that the number of units may be only two or three for the ordinary station. In some cases but a single unit may be used, as at the Searsburg plant (see Table 72), which is an automatic station.

Efficiency of Units at Part Load.—With low capacity factors—50 per cent or less, for a part of the 24-hr. period—a single unit operating at relatively low gate can carry the load. This will mean some loss of efficiency, as shown in Chap. IV, Fig. 77, but it should not greatly affect the average daily efficiency (unless the units are very large, as compared with load demand), as the output during this time is but a small part of the 24-hr. total. Furthermore, as previously stated, part-gate efficiency of medium- and low-specific-speed wheels, which would be used for medium- and high-head developments, is considerably better than that of the high-specific-speed wheels used for low heads. Water is always more valuable, and, hence, part-gate efficiency is more important in medium- and high-head developments.

One of the disadvantages of the fixed-blade propeller type of wheel, as has been stated, is the rapid decrease of efficiency at part gate, and, where it is used for low heads, this must be kept in mind in determining the size and number of units for the given or assumed load conditions. By including one or more of the Kaplan adjustable-blade wheel units in the installation, it is possible to handle the lighter loads efficiently.

After all, the plant efficiency under varying loads must depend considerably upon intelligence and care in operation. For any given load there will always be a best gate or gates for the wheel units, and the extent to which operation can meet this ideal will be of importance in effecting good plant efficiency.

TABLE 73.—RELATION OF SPECIFIC SPEED AND HEAD

Head, feet	Specific speed	Specific speed for propeller type
10- 20	110-100	190-185
20- 40	110- 80	185-165
40- 60	80- 70	165-145
60- 80	80- 60	145-115
80-130	60- 50	
130-200	50- 40	
200-400	40- 30	
400-800	30- 20	

Limitations of Specific Speed.—As explained in Chap. IV, for best results in wheel design a fairly definite relation between specific speed and head should be maintained, which may be summarized as shown in Table 73 on page 392.

While wheels as constructed and used have varied considerably from the relations given above, they correspond to the data shown in Fig. 81, Chap. IV, page 228.

In selecting wheel units, therefore, for any given head, this general limitation of specific speed must be first kept in mind.

Limitation in Generator Speeds and Sizes.—As further discussed in Chap. VIII, there are also certain limitations in generator speed to accord with the frequency and number of poles and often certain generator capacities which are normally obtainable. The selection of wheels and generators must, therefore, be more or less interdependent, and often turbines with some range in capacity and speed must be considered in different combinations which will give about the requisite total power. This may also affect the number of units.

In Table 74 are given data of ordinary capacities, speed, and voltage of standard three-phase, 60-cycle generators, which will be useful in preliminary station layouts.

In addition to these standard capacities, generators of other capacities are made "special" to meet particular requirements.

TABLE 74.—STANDARD VERTICAL WATER-WHEEL-DRIVEN GENERATOR CAPACITIES, SPEEDS, AND VOLTAGES (THREE PHASE, 60 CYCLES)

Capacities (in kilowatts at 0.8 power factor):

Below and at 2400 kw. (3000 kva.)

50, 60, 75, 100, 125, 150, 175, 200, 250, 300
350, 400, 450, 500, 600, 700, 800, 900, 1000,
1100, 1200, 1300, 1400, 1500, 1600, 1800, 2000, 2200, 2400

Above 2400 kw.

3,000, 4,000, 5,000, 6,000, 7,500, 10,000, 15,000, 20,000
25,000, 30,000, 35,000, 40,000, 50,000, 60,000, 70,000, 80,000

Speeds (in r.p.m.):

Below and at 2400 kw. (all capacities)

100, 112, 120, 128, 138, 150, 164, 180, 200, 225, 240
257, 300, 327, 360, 400, 514, 600, 720, 900

Above 2400 kw.

Speeds as above *except*

Add 80 r.p.m., all capacities

Omit 600, 720, 900 r.p.m., all capacities

Omit 450 and 514 r.p.m. for 60,000-kw. capacity and above

Omit 360 and 400 r.p.m. for 70,000-kw. capacity and above

Omit 320 and 327 r.p.m. for 80,000-kw. capacity

TABLE 74.—STANDARD VERTICAL WATER-WHEEL-DRIVEN GENERATOR CAPACITIES, SPEEDS, AND VOLTAGES (THREE PHASE, 60 CYCLES).—(Continued)

Voltages (in volts):

Voltage, below and at 2400 kw.	Range of Capacity, Kilowatts
2,400	50-2400
4,150	219-2400
6,900	312-2400
11,000	563-2400
13,800	750-2400
Above 2400 kw.	
2,400	2400- 7,500
4,150	2400- 15,000
6,900	2400- 25,000
11,000	2400-100,000
13,800	2400-100,000

Dimensions, weights, speeds, etc., of certain generators up to 25,000-kva. capacity are also given in Tables 78 and 79, Chap. VIII.

Examples of Tentative Selection of Wheel Units.

Example 1. Elements of Development.—Head 100 ft. Drainage area 500 sq. miles. Development to flow available one-third of the time or about 1.3 sec.-ft. per square mile or 650 sec.-ft. Generators to be three phase, 60 cycle, 2300 volts, with either three or four units. Specific speed should be about 55 (Chap. IV, Fig. 81, or Table 73).

1. With three units: Total wheel horse power about 6300; each unit = 2100 hp. For $N_s = 55$, $N = 380$; hence, a 1600-kw. generator at 360 r.p.m. may be used, requiring adjustment of wheel unit to 2300 hp., for which $N_s = 55$, which is satisfactory. For $N_s = 55$, $P_u = 0.0016$ (Chap. IV, Fig. 72) or a wheel diameter of about 38 in.

2. With four units: Each wheel unit about 1575 hp. $N = 440$; hence, a 1200-kw. generator at 360 r.p.m. may be used, with a 1700-hp. wheel. Revised value of $N_s = 47$, $P_u = 0.0012$, or a wheel diameter of about 37 in.

Summarizing:

For three units:

Wheel units, 38 in.	2300 hp., 360 r.p.m.
Generator units.	1600 kw. at 360 r.p.m.

For four units:

Wheel units, 37 in.	1700 hp., 360 r.p.m.
Generator units.	1200 kw. at 360 r.p.m.

Example 2.

Head = 25 ft. Discharge = 2000 sec.-ft.

$$\text{Total wheel horse power} = \frac{2000 \times 25}{8.8} \times 0.85 = 4800.$$

Wheel units, 1600 hp. (three units) or 1200 hp. (four units).

Generator capacity = 1100 or 850 kw.

Specific speed should be about 90.

1. With three units: $N = 126$ for which an 1100-kw. generator is available at a speed of 128 r.p.m.

2. With four units: $N = 145$; use a 900-kw. generator at 150 r.p.m., requiring a 1280-hp. wheel with $N_s = 96$ (or a 900-kw. generator at 138 r.p.m., requiring $N_s = 89$, may be used).

Approximate wheel sizes:

1. 1600 hp. = $0.038D^2 25^{3/4}$; $D^2 = 3370$; $D = 58$ in.

2. 1280 hp.: $D^2 = \overline{58}^2 \times \left(\frac{1280}{1600}\right) = 2680$; $D = 52$ in.

The assumed conditions also permit the use of a propeller-type wheel with $N_s = 140$ to 150 (see Chap. IV, Fig. 81).

1. With three units: $N = 200 \pm$; use an 1100-kw. generator at 200 r.p.m.

2. With four units: $N = 225 \pm$; use a 900-kw. generator.

Approximate wheel sizes:

1. 1600 hp. = $0.0019D^2 \times 25^{3/4}$; $D^2 = 6700$; $D = 82$ in.

2. 1280 hp.: $D^2 = \overline{82}^2 \times \left(\frac{1280}{1600}\right) = 5400$; $D = 74$ in.

These wheels may vary 5 to 10 in. in diameter, depending upon the particular kind of propeller type considered.

Summarizing:

For three units:

Wheel units, 58 in., 1600 hp., 128 r.p.m. or 82 in., 1600 hp., 225 r.p.m. (propeller type).

Generator units, 1100 kw. at 128 r.p.m. or 1100 kw. at 225 r.p.m. (for propeller-type wheel).

For four units:

Wheel units, 52 in., 1280 hp., 138 r.p.m. or 74 in., 1280 hp., 200 r.p.m. (propeller type).

Generator units, 900 kw. at 150 r.p.m. or 900 kw. at 225 r.p.m. (for propeller-type wheel).

The approximate data as determined in the two above examples will serve as a tentative basis for decision as to number of wheel units and for conference with wheel and generator manufacturers who may then furnish approximate dimension sheets along with a cost estimate, which will serve in the preliminary studies for the development.

Typical wheel-dimension sheets for three types of wheel settings are shown in Figs. 162 to 164 through the courtesy of the S. Morgan Smith Company, and appended are accompanying dimensions for several sizes of wheels. A preliminary layout sheet of the Allis-Chalmers Company for open wheel-pit setting is also shown in Fig. 165. Generator preliminary dimensions are given in a range of capacity in Table 78, Chap. VIII.

Specifications and Contract for Wheel Units.—After the results of the preliminary studies and conference with one or more manufacturers are available, definite specifications for wheel units may be made and proposals obtained from a number of wheel makers.

Specifications for wheel units should be framed to cover essential requirements, giving such information as is necessary to enable the wheel manufacturers to submit bids on the same general basis and, hence, amenable to comparison, but leaving them free to outline such details or special features of construction as they commonly employ. Such

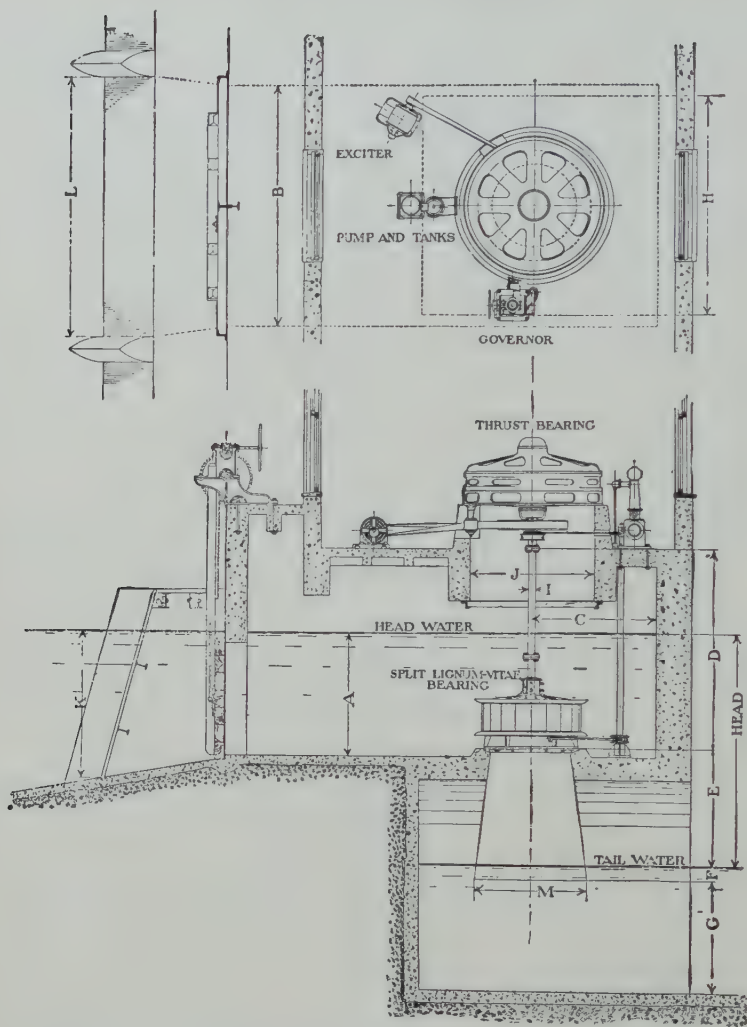
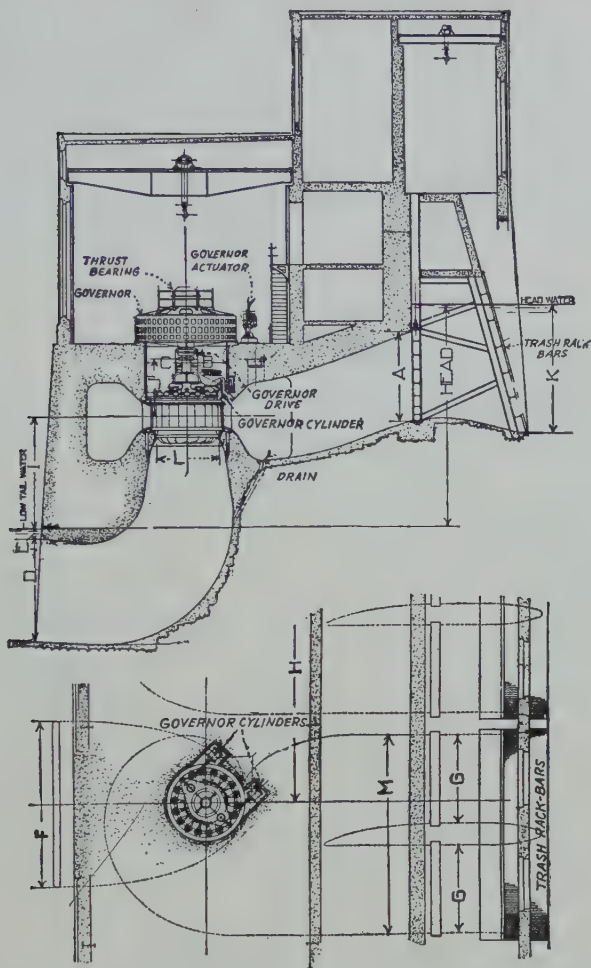


Fig. 162.—Open flume turbine setting. (S. Morgan Smith Company.)

Head, ft.	Diam., in.	Hp.	R.p.m.	A	B	C	D	E	F	G	H	I	J	K	L	M
25	36	625	200	"	"	"	"	"	"	"	"	"	"	"	"	"
25	48	1200	150	14-6	15-0	7-6	16-9	12-0	1-0	8-0	15-0	6 7/16	7-9	14-6	15-0	8-6
25	66	2225	112.5	14-0	20-0	10-0	17-6	12-0		8-6	20-0		10-0	14-0	20-0	13-0
				19-0	34-0	12-6	21-6	7-6		7-0	34-0		14-0	19-0	34-0	17-0

FIG. 163.—Concrete spiral-flume turbine setting. (*S. Morgan Smith Company.*)

Head, ft.	Diam., in.	Hp.	R.p.m.	A	B	C	D	E	F	G	H	I	J	K	L	M
50	39	1360	257	9-0		6-0	6-6	2-0	12-0		18-0	15-0	8-0	12-0	4-10	11-0
50	52	2412	200	12-0		8-0	8-6	2-6	16-8	7-0	23-0	15-0	9-0	15-0	6-3	
50	79	5120	133	16-0		11-6	12-6	3-3	24-0	11-0	30-0	15-0	10-0	22-0	8-10	

specifications should, therefore, include the following information and requirements:

- 1. Location of proposed plant and shipping point (often wheel equipment is required f.o.b. at nearest railroad freight station to the plant and hauling to the plant done by the general contractor).
- 2. Brief outline of the manner of development (as a dam located at the head of Union Falls, about 30 ft. high, with spillway level at an elevation of 700, with open canal about 600 ft. long, followed by individual penstock lines, each 6 ft. in diameter

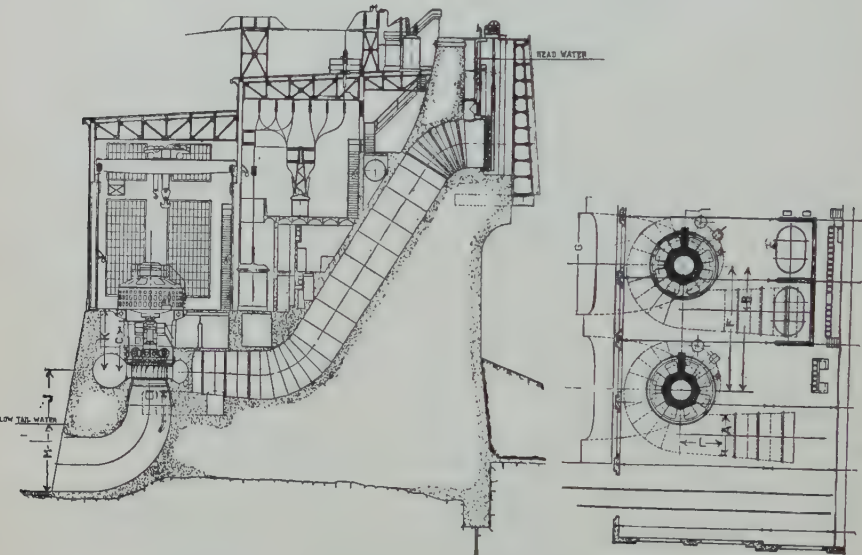


FIG. 164.—Plate-steel spiral-flume turbine setting. (S. Morgan Smith Company.)

Type	Diam., in.	Head, ft.	Hp.	R.p.m.	A	B	C	D	E	F	G	H	I	J	K	L
K-1	36	100	2,500	400	5-0	5-8	8-0			18-0	14-0	7-3	2-0	15-0	10-0	
	59	100	7,500	225	8-0	9-9	8-6	13½	12-0	24-0	17-0	10-0	2-0	10-0	12-0	10-0
	101	100	18,600	144	15-0	14-0	14-6			45-0	41-0	12-0	3-0	15-0	18-0	
F-1	42	400	10,000	600	4-0	5-2	8-6			17-0	10-3	5-3	2-0	18-0	10-0	6-6
	56	400	17,800	450	5-0	7-0	8-6			21-0	14-6	7-6	2-0	18-0	10-0	8-0
	73	400	28,000	360	6-0	8-5	12-0	17¾	11-0	24-0	16-0	9-0	2-0	15-0	12-0	10-0

and 500 ft. long, serving vertical wheel units with center at approximately an elevation of 615 and tailrace level at an elevation of 600).

3. Number and kind of wheel units and setting (as three vertical units, with plate-steel spiral-flume setting, and vertical flaring draft tube about 17 ft. long). Where a concrete draft tube is to be used, a short length of steel or iron tube about 3 or 4 ft. long is usually included in the wheel contract.

4. Power or capacity, head and speed of each unit under normal operating conditions (as to develop at full gate about 2150 hp. under a normal head of 100 ft., at a speed of 360 r.p.m. or to utilize a discharge of about 220 sec.-ft. at full gate under, etc.).

5. Range of head, if any (as maximum head 105 ft., minimum head 95 ft.).

6. Generator to be operated, capacity, characteristics, etc. (as to be direct connected with and operate a vertical, three-phase, 60-cycle, 2300-volt generator of about 1500-kw. maximum capacity at 0.80 power factor, at 360 r.p.m.).

7. Efficiency requested for full gate and full load and through the ordinary operating range, to be shown either by load-efficiency diagram or tabular statement.

8. Tests.—The bid should be accompanied by a Holyoke test sheet for a wheel of the size (as nearly as available) and type proposed. Provision may also be made for testing the wheel or a model runner after manufacture, at Holyoke, and often a test may be required in place when the plant is ready to operate.

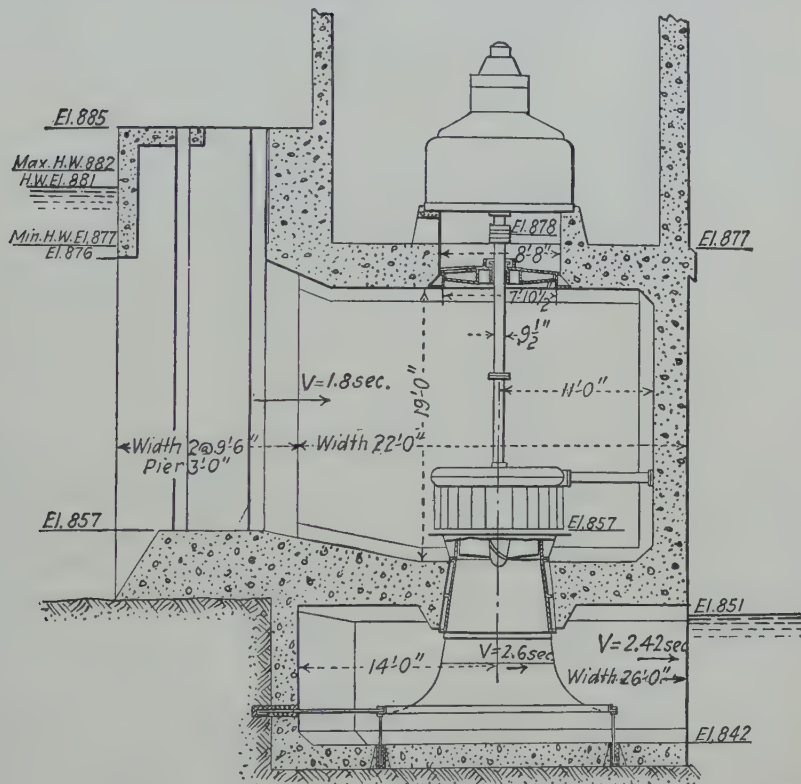


FIG. 165.—Open wheel-pit turbine setting—preliminary plan 1550 hp., 30 ft. head, 225 or 257 r.p.m. (Allis-Chalmers Manufacturing Company.)

9. Materials.—The material and type of construction of runner, flume or casing, speed ring, pit liner, curb and crown plates, and gate mechanism should be stated. Thus, depending on conditions, the runner may be cast iron, cast steel, bronze; the casing plate of cast iron or steel, or structural steel; the speed ring, pit liner, and curb and crown plates usually of cast iron. The gate wickets are usually of cast steel with rolled-steel stems and levers.

10. Governor.—The type, make, and description of governor and foot-pound capacity, and also the type of oil-pressure system proposed by the wheel maker should be stated, and time required for operating the turbine gates. The speed variation should be given for the WR^2 of the proposed generator for full, three-fourths, one-half, and one-fourth load change.

11. Weight of entire wheel and of heaviest part.

12. Detail Plans.—Accompanying the bid should be a detailed plan of a wheel unit as nearly as may be like that proposed. As soon as practicable after award of contract, the successful bidder should supply a detailed plan of the wheel to be furnished.

13. Time of shipment from date of awarding contract.

14. Parts that may be required at an early date, such as the curb ring, draft tube and speed ring, should be noted. Additional details of construction, etc., to be included in specifications of manufacturer accompanying his bid.

15. Price bid, including also a price per day for services of erector. It should also be stated that wheel efficiency as well as price will be considered in awarding the contract. To make these conditions more definite, sometimes a value of power per kilowatt-hour is stated and expected capacity factor, so that a definite allowance may be made in the comparison of efficiency.

16. Unsatisfactory Completion of Contract.—In case the wheel units prove unsatisfactory after erection or fail to reach the guaranteed efficiency, they may be rejected or required to be made satisfactory at the expense of the manufacturer.

The approximate wheel plan furnished by the manufacturer with his proposal will usually serve as a tentative basis for proceeding with the power-house design, when the wheel contract is let. The actual wheel-detail plan, or provisional copy of it, will usually be available before it is necessary to decide upon the exact layout.

POWER-HOUSE SUBSTRUCTURE

Spacing of Units.—For the usual type of vertical unit, as commonly arranged, the spacing of the units will generally be fixed by the necessary width of water passages leading to and from the wheel, *viz.*, the flume (open or closed) above the wheel, and the draft tube and tailrace below the wheel. Frequently with open wheel-pit settings, the necessary flume width will be greater than that of the draft tube. For concrete or steel flumes usually the necessary width of draft tube and tailrace will fix the spacing of units. For high heads and relatively small discharge, where large-sized units are used, the necessary minimum clearance of 5 to 10 ft. or more between generators may determine the unit spacing.

For horizontal units, where they are arranged with main shaft normal to the long axis of the power house, the same general conditions govern the unit spacing as with vertical units; the same width of water passage is required as for a given capacity and speed; and the overall width of a horizontal generator is but little more than the overall diameter of a vertical generator. Where the units are arranged with main shaft parallel to the long axis of the power house, as in the case of the White River (Washington) plant, the unit spacing will be fixed by the length of the wheel and generator.

In the case of the White River plant, each unit consists of a 22,500-hp. horizontal wheel and generator operating at 360 r.p.m. under a head of 480 ft., occupying a total length of about 35 ft. parallel to the long axis

of the station, and the units are spaced about 37 ft. center to center of wheels. A thesis study of this plant made under the direction of the author by A. H. Crossman, Massachusetts Institute of Technology, 1923, indicated that with vertical wheel units, allowing 6-ft. clearance between generators and speed of 400 r.p.m., the unit spacing for the same powered units could be 24 ft.

The relative areas and volumes of power house required were as follows:

WHITE RIVER PLANT—POWER-HOUSE AREA AND VOLUME—HORIZONTAL *vs.* VERTICAL UNITS

Setting	Unit		Power house			
	Size, horse power	Speed, revolutions per minute	Area, square feet	Volume, cubic feet	Square feet per horse power	Cubic feet per horse power
Horizontal . .	22,500	360	15,400	678,000	0.68	30
Vertical	22,500	300	6,500	320,000	0.29	14

This comparison shows clearly the advantage of the vertical unit in respect to compactness of station layout. The unit areas and volumes should be compared with those given in Table 104, Chap. XII.

Power-development Data.—In Table 75 are given fairly complete data of turbine characteristics, dimensions, velocities, etc., and other features of power house and intake of 47 hydroelectric developments, compiled by and furnished through the courtesy of the J. G. White Engineering Corporation, covering a range of head from 13 to 849 ft. and including many plants in this country. This table has already been referred to; it served as a basis for constructing Figs. 81 and 88, Chap. IV, and should be carefully studied.

Addition to this table has been made (1933) of data relating to 21 more plants including some of the more important installations made since 1925, of which five have propeller-type wheels.

A comparison of line 13, spacing of units, in Table 75 with lines 22 and 23, entrance to scroll, and line 34, total width of draft tube, indicates that in only three of the high-head plants (Nos. 43, 47, and 68) the spacing of units was fixed by necessary clearance of generators; usually this spacing was determined by the total width of the draft tube at its outlet and necessary thickness of division walls.

Open Flume or Wheel Pit.—As stated in Chap. IV, the open-flume setting is used only for low heads (up to 20 ft. usually, and to about 30 ft. as a maximum) and, hence, it is used for concentrated falls or occasionally a development with short canal. The general arrangement of an open-flume setting is shown in Fig. 165, for a simple rectangular flume.

DATA ON LARGE HYDRO

TURBINES		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
1	Horizontal shaft	11.5	17	17	21	21	26	31	36	41	46	51	56	59	59	59	59	59	59	59	59	59	59
2	Rated Capacity (K.F.)	1000	2000	3000	4500	6000	9000	12000	15000	18000	21000	24000	27000	30000	33000	36000	39000	42000	45000	48000	51000	54000	57000
3	Minimum Capacity (K.F.)	1600	1850	1283	1315	1020	1030	1230	1260	1105	1700	2000	1717	1680	1600	1500	1550	1500	1500	1500	1500	1500	1500
4	Maximum Capacity (K.F.)	1232	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212	1212
5	Speed (R.P.M.)	80	62	68.8	75	100	55.6	37.7	80	118.8	90	116	83.3	64.1	58.3	162	69.7	76.8	61.7	53.8	80	61.7	80
6	Specific Speed (R.H.)	11	78.5	89.0	65.8	62.7	62.3	16.8	70.2	31.2	46.4	39.2	42.8	44.1	45.3	15.5	31.3	30.0	29.0	28.0	27.0	26.0	25.0
7	Type Runner	Prop.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.	Franc.
8	Scroll Case Material	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast	Cast
9	Drift Tube Type	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic	Hydraulic
10	Drift Tube Seal with Gaskets	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	Manufacturer	A-C	A-C	M.S.M.	3.6.6.	1.8.M.	1.8.M.	1.8.M.	1.8.M.	A-C	3.6.6.	3.6.6.	1.8.M.	M.S.M.	A-C	1.8.M.	1.8.M.	M.S.M.	A-C	1.8.M.	1.8.M.	1.8.M.	1.8.M.
SUBSTRUCTURE DIMENSIONS																							
12	Spanning of Units	65'-0"	36'-0"	35'-0"	27'-0"	50'-0"	66'-0"	30'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
13	Spanning of L.S.R.	19'-6"	—	61'-0"	16'-0"	18'-6"	18'-6"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
14	Foundation in Concrete at Deck level	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
15	Foundation in Operating Floor	18'-6"	16'-0"	16'-0"	17'-0"	17'-0"	24'-0"	16'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
16	Foundation in Bottom of Draft Tube	23'-0"	23'-0"	23'-0"	23'-0"	23'-0"	23'-0"	23'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
17	Foundation in Draft Tube Discharge	23'-0"	23'-0"	23'-0"	23'-0"	23'-0"	23'-0"	23'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
18	Foundation in Upstream Area A	24'-0"	7'-0"	6'-0"	5'-0"	3'-0"	4'-0"	8'-0"	9'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"
19	Low Tail Water to High Tail Water	25'-0"	6'-0"	6'-0"	27'-0"	3'-0"	5'-0"	20'-0"	21'-0"	42'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"	6'-0"
20	Drift Tube	18'-0"	16'-0"	15'-0"	14'-0"	12'-0"	11'-0"	10'-0"	10'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"	12'-0"
TURBINE DIMENSIONS																							
21	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
22	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
23	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
24	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
25	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
26	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
27	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
28	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
29	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
30	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
31	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
32	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
33	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
34	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
35	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
36	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
37	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
38	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
39	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
40	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
41	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
42	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
43	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
44	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
45	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
46	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
47	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
48	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
49	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
50	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
51	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
52	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
53	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
54	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
55	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
56	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
57	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
58	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
59	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
60	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
61	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
62	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
63	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
64	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
65	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
66	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
67	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
68	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
69	Entrances to Scroll	19'-6"	19'-6"	20'-0"	20'-0"	20'-0"	20'-0"	22'-0"	—	—	—	—	—	—	—	—							

PREPARED BY THE J.G. WHITE ENGINEERING CORPORATION

TABLE 75.—(Continued.) ADDITION FOR 1933

DATA ON LARGE HYDRO-ELECTRIC DEVELOPMENTS

TURBINES		40	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68
1	Normal head	7	27	32	17	30	53	60	—	271	89	61	710	2	124	134	730	2915	3803	330	4560	3000
2	Rated Capacity (hp)	800	3600	2,050	11,500	9,000	42,500	20,500	10,000	8,000	50,000	50,000	43,000	22,000	21,500	34,000	14,000	20,000	13,000	78,500	5,500	3,500
3	Rated Discharge CFS	1100	1250	1400	35,000	18,500	4,000	3,450	—	1125	6,300	3,300	4,700	2,600	1,650	2,450	3,550	3,250	1,000	864	1,25	2,150
4	Minimum Capacity, hp	—	—	22,500	14,000	—	4,700	—	—	—	—	34,000	—	—	—	36,000	—	—	—	—	—	—
5	Maximum Discharge CFS	—	—	7,350	4,000	—	9,500	—	—	—	—	6,000	—	—	—	3,800	—	—	—	—	—	—
6	Speed R.P.M.	80	150	100	1125	1091	851	100	1636	616	84	1125	150	150	150	1383	103	107	225	300	25	2350
7	Specific Speed (Ft-lb)	196.1	145	191	121	60	152	72	70	64	70	61	512	55	62	347	553	60	264	24	24	24
8	Type, Runner	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis	Francis
9	Scroll Case Material	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete
10	Drift Tube Type	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly	Manly
11	Total Wt. in Tons (not with Governor)	416 T.	—	283,000	600 T.	170 T.	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
12	Manufacturer	Camp	S.M.S.	A.C.	A.C.	A.C.	A.C.	A.C.	Camp	Heppner	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.
SUBSTRUCTURE DIMENSIONS																						
13	Spacing of Units	48' 0"	35' 0"	60' 0"	50' 0"	40' 0"	44' 9"	—	—	6' 0"	72' 0"	54' 0"	48' 0"	30' 0"	45' 0"	45' 0"	60' 0"	—	40' 0"	40' 0"	55' 0"	62' 6"
14	Distance to L.T.M.	6' 0"	7' 2"	4' 6"	9' 6"	3' 3"	16' 0"	13' 6"	11' 0"	10' 0"	16' 0"	1' 11"	15' 0"	10' 0"	8' 0"	13' 0"	—	—	—	—	—	—
15	Distance to Center of Gravity	15' 0"	10' 0"	27' 0"	43' 6"	18' 0"	31' 3"	31' 6"	35' 0"	16' 0"	29' 0"	29' 6"	35' 0"	20' 0"	17' 0"	23' 0"	21' 0"	17' 0"	14' 0"	16' 0"	16' 0"	16' 0"
16	Distance to Operating Floor	11' 0"	10' 0"	27' 6"	43' 6"	18' 0"	31' 3"	31' 6"	35' 0"	16' 0"	29' 0"	29' 6"	35' 0"	20' 0"	17' 0"	23' 0"	21' 0"	17' 0"	14' 0"	16' 0"	16' 0"	16' 0"
17	Distance to Bottom of Draft Tube	22' 0"	20' 0"	43' 0"	33' 0"	22' 0"	48' 9"	49' 4"	43' 0"	21' 0"	41' 0"	41' 6"	51' 0"	22' 0"	22' 0"	22' 0"	22' 0"	22' 0"	22' 0"	22' 0"	22' 0"	22' 0"
18	Distance to Draft Tube Discharge	18' 0"	31' 6"	43' 0"	27' 0"	29' 0"	72' 0"	100' 0"	3' 0"	22' 0"	56' 6"	48' 0"	30' 0"	36' 0"	41' 0"	36' 0"	32' 0"	19' 0"	23' 0"	56' 6"	56' 6"	56' 6"
19	Distance to Upstream Face D.T.	16' 6"	9' 6"	12' 0"	25' 0"	7' 9"	9' 6"	13' 6"	16' 6"	9' 6"	16' 6"	9' 6"	16' 6"	7' 0"	17' 0"	6' 0"	6' 0"	5' 0"	4' 0"	12' 6"	4' 0"	6' 6"
20	Low Tail Water to High Tail Water	—	24'	54' 0"	65' 0"	—	23' 6"	31'	—	20' 0"	21' 0"	30' 0"	—	24' 0"	—	15' 4"	4' 0"	—	—	19' 0"	4' 0"	21' 0"
21	P.T. Diameter	15' 0"	13' 6"	25' 0"	20' 3"	15' 0"	24' 5"	17' 5"	18' 0"	10' 0"	25' 0"	18' 0"	18' 0"	14' 6"	16' 6"	16' 0"	22' 0"	16' 0"	12' 0"	13' 0"	13' 0"	13' 0"
TURBINE DIMENSIONS																						
22	Entrance	Width of Guide Vanes										Width of Nozzle										
23	To Scroll	15' 1/4"	18' 1/2"	20' 1/2"	20' 1/2"	20' 1/2"	20' 1/2"	20' 1/2"	20' 1/2"	20' 1/2"	20' 1/2"	11' 0"	25' 6"	27' 2"	22' 0"	14' 0"	13' 0"	16' 0"	14' 0"	9' 0"	7' 0"	7' 0"
24	Exit to Inlet	2' 6"	0' 0"	2' 0"	0' 0"	11' 0"	—	4' 1"	14' 0"	26' 0"	24' 0"	—	18' 0"	26' 0"	24' 0"	—	18' 0"	26' 0"	24' 0"	—	18' 0"	26' 0"
25	Exit to Inlet of Scroll Case	13' 0"	12' 0"	31' 4"	28' 0"	16' 0"	28' 0"	50' 0"	14' 0"	18' 0"	21' 0"	17' 0"	—	19' 0"	21' 0"	—	19' 0"	21' 0"	—	19' 0"	21' 0"	21' 0"
26	Scroll Quadrant T. from E	22' 0"	15' 6"	26' 0"	20' 0"	20' 0"	20' 0"	20' 0"	18' 0"	36' 0"	33' 0"	—	24' 0"	36' 0"	33' 0"	—	24' 0"	36' 0"	33' 0"	—	24' 0"	36' 0"
27	" " " "	16' 6"	10' 8"	19' 0"	17' 0"	16' 6"	22' 0"	15' 0"	34' 0"	35' 0"	24' 6"	19' 0"	16' 3"	—	20' 0"	24' 6"	—	20' 0"	24' 6"	—	20' 0"	24' 6"
28	" " " "	—	15' 6"	26' 0"	14' 0"	—	—	—	13' 0"	25' 0"	29' 3"	—	—	—	—	—	—	—	—	—	—	—
29	" " " "	—	10' 8"	—	—	—	—	—	8' 0"	21' 2"	23' 6"	12' 6"	13' 0"	12' 6"	—	12' 6"	13' 0"	—	12' 6"	13' 0"	—	12' 6"
30	Diameter Inlet Stop Flange	—	14' 0"	25' 6"	23' 0"	13' 6"	25' 0"	9' 6"	9' 0"	23' 0"	23' 0"	13' 4"	12' 6"	—	13' 4"	12' 6"	—	13' 4"	12' 6"	—	13' 4"	12' 6"
31	Height Guide Vane Case	4' 0"	3' 6"	8' 6"	8' 0"	7' 0"	7' 5"	5' 4"	4' 8"	24' 0"	7' 0"	6' 0"	—	3' 6"	3' 0"	2' 6"	3' 0"	—	3' 0"	2' 6"	—	3' 0"
32	Distance to Center Discharge	6' 0"	21' 0"	8' 6"	3' 0"	9' 0"	8' 9"	3' 6"	5' 0"	3' 6"	7' 0"	7' 6"	—	3' 0"	3' 0"	3' 0"	3' 0"	—	3' 0"	3' 0"	—	3' 0"
33	Diameter of Center Discharge	11' 6"	9' 6"	17' 0"	10' 0"	10' 0"	10' 0"	12' 0"	13' 6"	8' 9"	16' 6"	17' 0"	13' 0"	11' 0"	10' 6"	11' 0"	16' 6"	13' 0"	6' 6"	7' 4"	11' 0"	11' 0"
DRAFT TUBE																						
34	Maximum Width D.T.	39' 0"	28' 0"	59' 0"	54' 0"	35' 6"	53' 3"	31' 0"	—	36' 6"	56' 6"	—	—	—	39' 0"	28' 0"	—	—	32' 0"	17' 0"	35' 4"	34' 0"
35	Center Pier Width	20' 0"	3' 0"	6' 0"	6' 0"	—	4' 6"	—	—	20' 0"	5' 6"	—	—	—	5' 0"	4' 0"	—	—	3' 0"	—	—	—
36	D.T. Discharge Height	13' 0"	6' 6"	16' 0"	10' 0"	10' 0"	13' 0"	20' 0"	14' 0"	11' 6"	22' 6"	21' 6"	19' 0"	9' 3"	12' 0"	13' 0"	9' 0"	8' 0"	11' 0"	14' 0"	14' 0"	
37	D.T. Discharge Wet Width	39' 0"	28' 0"	59' 0"	54' 0"	35' 6"	53' 3"	31' 0"	—	36' 6"	56' 6"	—	—	—	39' 0"	28' 0"	—	—	32' 0"	17' 0"	35' 4"	34' 0"
38	Shape of D.T.	000	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
39	Bottom D.T. to Bottom Bell	—	—	—	—	—	—	—	—	3' 6"	9' 6"	5' 3"	4' 6"	—	5' 3"	—	—	—	4' 5"	5' 0"	—	—
SUPERSTRUCTURE																						
40	Exit Outside P.T. (Downstream)	19' 0"	31' 0"	27' 0"	27' 0"	22' 3"	34' 6"	60' 0"	28' 0"	23' 0"	39' 0"	40' 0"	34' 0"	22' 6"	41' 0"	35' 6"	19' 6"	26' 0"	19' 0"	23' 0"	78' 0"	78' 0"
41	Exit Outside P.T. (Upstream)	11' 6"	14' 0"	23' 0"	21' 0"	11' 4"	28' 6"	16' 0"	20' 0"	15' 0"	33' 3"	35' 0"	23' 0"	29' 0"	19' 0"	24' 6"	81' 6"	21' 0"	23' 0"	30' 0"	46' 6"	46' 6"
42	Overhead Tank to Top Center Bell	23' 0"	20' 6"	18' 0"	42' 0"	24' 6"	42' 0"	34' 0"	32' 0"	23' 3"	—	31' 0"	41' 0"	28' 0"	32' 0"	30' 0"	41' 0"	46' 0"	35' 0"	50' 0"	78' 0"	78' 0"
43	Top Center Bell to Top Road Cross	8' 0"	7' 0"	9' 6"	10' 0"	8' 6"	23' 0"	10' 0"	11' 0"	5' 6"	—	12' 0"	13' 0"	10' 0"	9' 0"	10' 0"	10' 0"	10' 0"	10' 0"	13' 6"	13' 6"	13' 6"
44	Crane Capacity, Main	25' 0"	60' 0"	125' 0"	202' 0"	212' 5"	95' 0"	100' 0"	—	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"	21' 0"
45	Crane Capacity, Auxiliary	—	15' 0"	—	—	—	4' 0"	—	—	—	—	25' 0"	13' 0"	25' 0"	8' 0"	15' 0"	—	—	11' 0"	30' 0"	30' 0"	30' 0"
INTAKE																						
46	Slope of Rock Bars	20' 1"	14' 0"	Vertical	Vertical	6' 0"	2' 30"	18' 0"	Slight Slope	18' 0"	—	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	13' 0"	13' 0"
47	Size of Rock Bars	4' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"	3' 1"
48	Rock Bar Spacing C to C	3' 6"	12' 0"	—	—	—	7' 0"	—	—	—	—	4' 0"	4' 0"	3' 0"	3' 0"	3' 0"	3' 0"	3' 0"	3' 0"	3' 0"	3' 0"	3' 0"
49	Rock Bar Supports	15' 0"	15' 0"	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete
50	Allowed Head on Supports	20' 3"	5' 0"	—	—	Full	30' 0"	43' 0"	—	—	—	5' 0"	—	—	—	20' 0"	—	—	20' 0"	20' 0"	20' 0"	20' 0"
Type of Gates																						
51	Size of Gates	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical	Vertical
52	Head on Bot. of Gates	23' 0"	19' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"
53	Head on Bot. of Gates	23' 0"	19' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"
54	Head on Bot. of Gates	23' 0"	19' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"	23' 0"
WATER VELOCITIES AT RATED DISCHARGE																						
55	Rocks Gross N.H.W.	2.8	1.9	2.46	2.5	2.2	3.64	2.0	1.2	2.8	—	2.0	1.9	2.66	1.3	—	1.2	1.0	1.0	1.0	1.0	1.0
56	Rocks Gross L.H.W.	—	—	2.46	—	—	—	1.8	—	—	—	4.2	1.9	—	2.1	—	—	1.0	1.0	1.0	1.0	1.0
57	Gates	—	3.8	6.1	2.00	9.0	5.05	3.2	5	9.3	—	7.2	12.3	5.3	5.7	—	10.4	5.4	12.1	12.9	9.1	9.1
58	Flow Line	—	—	—	—	—	—	—	—	—	—	—	13	—	—	—	—	11.2	19.1	13.3	50	50
59	Flow Capacity	—	—	—	2.00	—	—	—	—	—	—	10	12.25	12.45	11.9	—	15.6	14.4	16.9	17.7	17.7	17.7
60	Entrance to Scroll Case	175	37	61	200	7.6	7.0	7.1	12.1	12.7	9.3	1.0	14.0	12.45	15.9	21	15.6	22.4	29.2	29.9	29.9	29.9
61	Runner Discharge (Not Comp.)	10.6	17.6	30.0	20.0	32.6	12.2	—	10.5	24.2	23.3	31.5	22.7	—	25.7	25.0	21.6	—	—	25.7	25.7	25.7
62	Exit from Draft Tube	26	6.4	4.2	5.2	8.34	5.0	—	3													

Occasionally the downstream corners of the flume are filled and rounded, and sometimes a nose pointing toward the wheel is provided to give smoother flow. The design and arrangement of racks and gates are discussed in Chap. IX. The gross section just upstream from the racks should be sufficient to give a velocity of flow of about $0.03\sqrt{2gh}$, and at the gates usually $V = 0.07\sqrt{2gh}$ (see Table 81, Chap. IX) and about the same velocity in the flume.

Frequently, to give sufficient waterway and, at the same time, not require too large gates, one or more piers will be required, which should have a rounded or cutwater upstream end and be brought to an approximate taper point at their downstream end in the flume. These piers, depending upon their height and span between, will be from 1.5 to 3 ft. in thickness. The flume walls will be somewhat thicker, as they must be designed for possible water load on one side alone.

The wall and floor of the wheel pit are usually quite heavily loaded. Use of low concrete stresses is recommended, and no reliance should be placed on concrete taking tension, steel being used in such cases at half the usual allowable unit stress of 16,000 lb. per square inch. Very careful attention must be paid to the design of joints for water-tightness, and water stops should always be provided and placed in such a way as to protect the reinforcement. Where possible, the flume should be designed as a monolithic structure, at least up to the maximum water level, although preferably the whole structure should be of monolithic character.

The flume roof is commonly the power-house floor and designed accordingly, to carry the runner, the thrust on the runner, and the generator, as well as a uniform live load such as would be caused by placing the heaviest piece of equipment anywhere on the floor.

A preliminary layout for open-flume wheel setting, with one pier, as furnished by the Allis-Chalmers Company, is shown in Fig. 165, and velocities are shown at various points in flume, draft tube, and tailrace.

Concrete Scroll Flume.—As the concrete approach flume is likely to fix the unit spacing, it should be kept as small in width as proper design will allow. Its general dimensions adjacent to the wheel, form of spiral, etc., will usually be fixed by the wheel manufacturer, but the details of entrance, piers, nose, etc., must be designed by the engineer to accord with the general layout.

Practice varies with respect to the dimensions and general arrangement of the concrete flume, but a study of the upper plot on Fig. 88, Chap. IV, indicates that most of the points for concrete flumes (shown as filled circles) are consistent with $v = 0.06$ to $0.08\sqrt{2gh}$, and a value of $v = 0.07\sqrt{2gh}$ for velocity at entrance to the concrete flume may be taken as common.

The horizontal arrangement of spiral around the wheel should be such that no additional distance between units will be required beyond that required at entrance.

Figure 166 shows a common arrangement of concrete flume for a wheel of large capacity as used at the Turners Falls plant (see also Fig. 298, Chap. XIII). At entrance to the flume the velocity is about $0.08\sqrt{2gh}$ with spacing of end piers 37 ft. 0 in., and the spiral is arranged consistent with this same dimension for unit spacing; the center line of the flume passes through the center of the unit. The nose N makes an angle of about 285 deg. with the longitudinal axis of the station measuring from

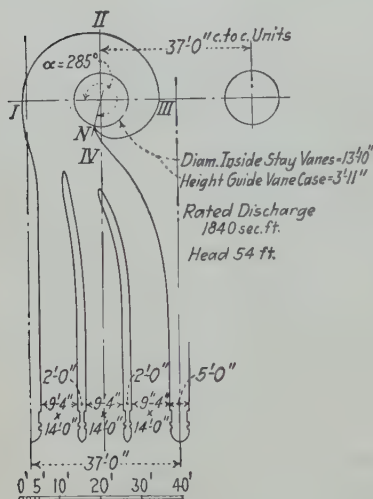


FIG. 166.—Concrete spiral flume for large wheel—Turners Falls plant of Turners Falls Power and Electric Company.

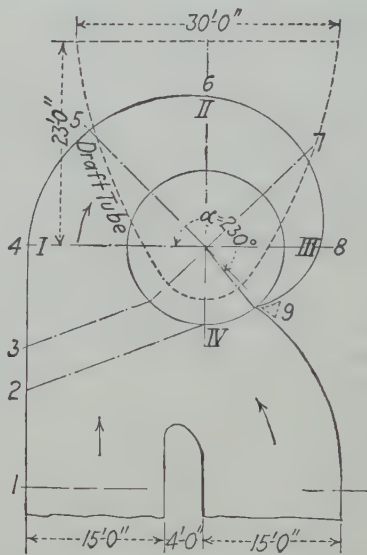


FIG. 167.—Concrete spiral flume and draft tube—Vischer Ferry and Crescent plants, New York State Canal System.

section I. For a smaller unit the same general scheme may be employed, omitting one or both of the auxiliary piers, if this can be done and still provide suitable gate dimensions and reasonable spans at a lesser cost, taking into account the gain in head with fewer piers.

The nose angle α may be made somewhat larger, requiring, however a greater flume width at section III. It may also be reduced to a minimum of about 225 deg. where the curvature of the side of the approach flume has to be such that a tendency to eddy formation will result.

A simple arrangement of concrete flume for the 4000-hp. wheel units of the Vischer Ferry and Crescent plants of the New York State Canal System, which operate under a head of 26.5 ft., is shown in Fig. 167. For a full-gate discharge of 1560 sec.-ft. the following velocities occur:

Point	Velocity, feet per second	$\frac{v}{\sqrt{2gh}}$
Racks.....	2.84	0.07
Flume entrance.....	3.25	0.08
Runner discharge.....	13.8	0.33
Draft-tube exit.....	3.15	0.076

The nose angle α is about 230 deg., and the flume entrance has one pier.

The details of the scroll flume are given in Table 76 (see also Fig. 299, Chap. XIII).

TABLE 76.—CONCRETE SCROLL FLUME—VISCHER FERRY AND CRESCENT PLANTS—
DIMENSIONS

Section (Fig. 167)	Width, feet	Mean height, feet	Area, square feet	Discharge past section, second- feet	Velocity, feet per second
1	30	16	480	1560	3.25
2	29	16	464	1387	2.99
3	20	15.7	314	1192	3.80
4	11.5	14.8	170	997	5.86
5	9.4	14.0	132	802	6.08
6	8.3	11.6	96	607	6.32
7	6.4	9.8	63	412	6.54
8	4.0	7.8	31	217	7.00
9	0.0	0.0	0	0	

Another general arrangement of concrete flume is shown in Fig. 168 (which is the arrangement of concrete flume at the Wilson dam), where with two auxiliary piers one of the subpassages is brought to the upstream side of the speed ring by an auxiliary nose, and its discharge kept distinct from the remaining flow through the other subpassages, which unite and enter the main scrollway. In this case the nose N should have α not less than about 225 deg. for reasons previously stated, while the nose N' will be in a position consistent with the fractional part of the discharge taken by the subpassage. In Fig. 168 this is $Q/3$ and there is consequently a central angle between noses of 120 deg. Four equal subpassages might also be used, with their central angle 90 deg., resulting in an auxiliary nose, less drawn out and thin as compared with that in Fig. 168 and, therefore, somewhat easier to construct and maintain.

The vertical sections of a concrete scroll case will, where conditions permit, normally be symmetrical, or nearly so, about the horizontal center

line of the wheel unit, as illustrated by the vertical section through the wheel unit of the Turners Falls plant (Fig. 298, Chap. XIII). The vertical side wall joining the horizontal roof and floor by curved fillet and straight slopes from roof and floor to speed ring in this case is also typical.

Theoretically, a circular cross section of the scroll case would be most efficient hydraulically, but with large units this would so widen the water passage as to require additional distance between units. Hence, the height of scrollway is usually greater than the width.

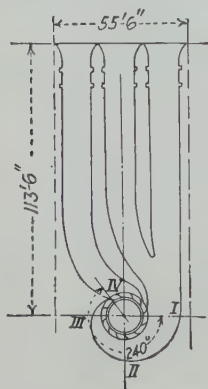


FIG. 168.—Concrete spiral flume at Wilson Dam.

For various reasons, but chiefly on account of head, it may be desirable to depart from the symmetrical arrangement of vertical sections in the concrete scroll. With a low head the elevation of the roof of the scroll may be fixed by the necessary depth below head-water level for a suitable water seal. This may require a greater depth of spiral flume below the wheel center line than above, in order to maintain the proper velocities in the flume. Such an arrangement is shown in Fig. 169, a cross section of power house and wheel unit, etc., at the Skowhegan, Me., development of the Central Maine Power Company, where a head of 32 ft. is utilized.

Conversely, where sufficient head is available for water seal, the greater portion of the scrollway cross section may be kept above the wheel center line, thus saving excavation, as shown in Fig. 170, a cross section of the wheel units, flume, and draft tube of the Great Falls development of the Manitoba Power Company, utilizing 56 ft. of head (see also Plant 15 in Table 75).

Where a quarter-turn draft tube is used, care must be taken to see that the floor or inner slope of the scroll case does not approach too near to the draft tube in the vertical plane through the center line of draft tube and wheel unit. The necessary clearance at this point for suitable strength of masonry may affect the shape and arrangement of the scroll case.

A typical arrangement of concrete scroll flume with an elbow draft-tube setting is shown in Fig. 299, Chap. XIII, for the Crescent plant. The details of this flume are given in Table 76 and a sectional plan in Fig. 167. Note that it is symmetrically arranged with reference to a horizontal line through the center of the unit.

Steel Scroll Flume.—The use of the concrete scroll flume is necessarily confined to low heads, usually not over 60 ft. The complicated form work and reinforcement required for the concrete flume make it expensive, with the result that plate-steel scroll flumes are now being used

with economy for heads as low as 40 ft. The upper limit of use of the plate-steel flume is about 350 ft. (as at the Davis Bridge plant of the New England Power Company). For greater heads cast-steel spiral casings are in general use.

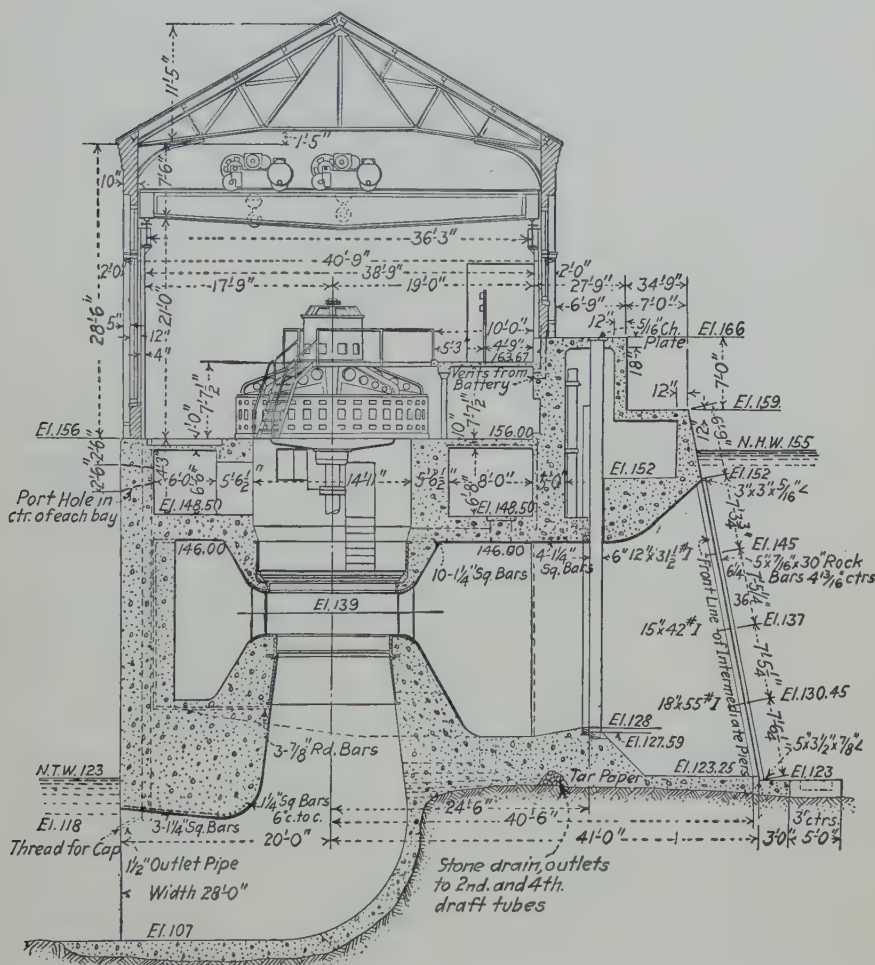


FIG. 169.—Cross section through wheel unit—Skowhegan, Me., development of Central Maine Power Company.

Practice is somewhat variable as to velocity at scroll entrance to steel flumes, as may be seen by reference to Fig. 88, Chap. IV; it varies from about $0.15\sqrt{2gh}$ for low heads to about $0.12\sqrt{2gh}$ for high heads.

Plate-steel Scroll Flumes or Casings.—For vertical units the penstock generally approaches the wheel on its horizontal center line but with the end of the penstock (or scroll entrance) offset laterally so that the far

wall of the penstock becomes the outside wall of the spiral case. Usually the scroll entrance is somewhat smaller in diameter than the penstock, so that a tapering transition section of pipe is required. Some type of penstock gate, usually of the wicket or plunger type, is normally placed in the penstock line near or at the taper section.

The steel scroll case is of circular cross section at its entrance, remaining approximately circular in section for some distance around the spiral but gradually becoming oval in shape as the section diminishes in size, in order to fit the speed-ring opening, which is of constant height around the wheel.

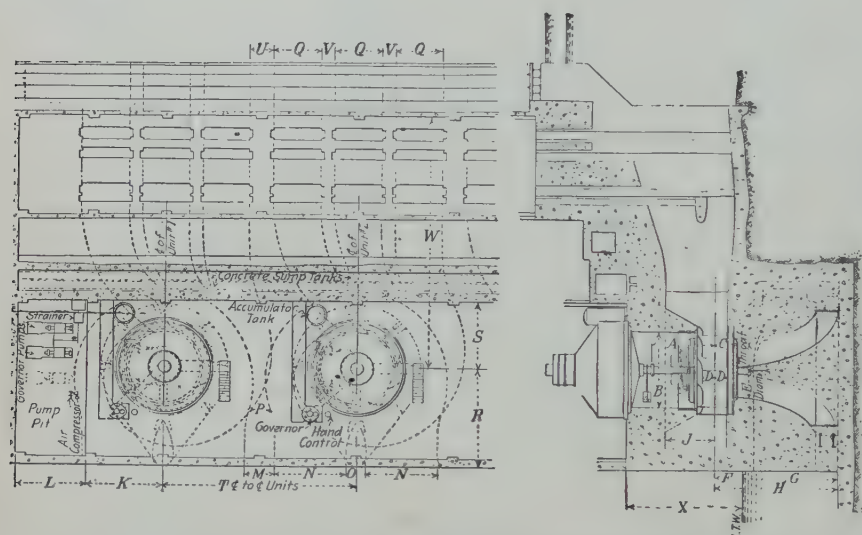


FIG. 170.—Arrangement of wheel units—Great Falls Development of Manitoba Power Company.

Details of a plate-steel scroll case as used for the wheel units at the Estes-Rainbow plant, New York, are shown in Fig. 171. These wheel units are 33 in., type K-1, S. Morgan Smith, operating at 400 r.p.m. under a head of 89 ft. At full gate they utilize about 220 sec.-ft., for which at scroll entrance (5 ft. 0 in. in diameter) the velocity is about 11 ft. per second or $v = 0.145 \sqrt{2gh}$. A taper section (not shown in Fig. 171) joins the scroll case and 6-ft. diameter penstock, the latter having at its end a motor-operated wicket penstock gate.

The method of assembling a plate-steel scroll casing is shown in Fig. 172, the various sections being first bolted into position and then finally riveted. The hole for a manhole entrance to the casing appears at the right.

Cast-steel Scroll Casing.—For heads above about 250 to 350 ft., depending upon the size of wheel, the plate-steel scroll casing becomes subject to considerable deformation, unless backed up with a heavy mass

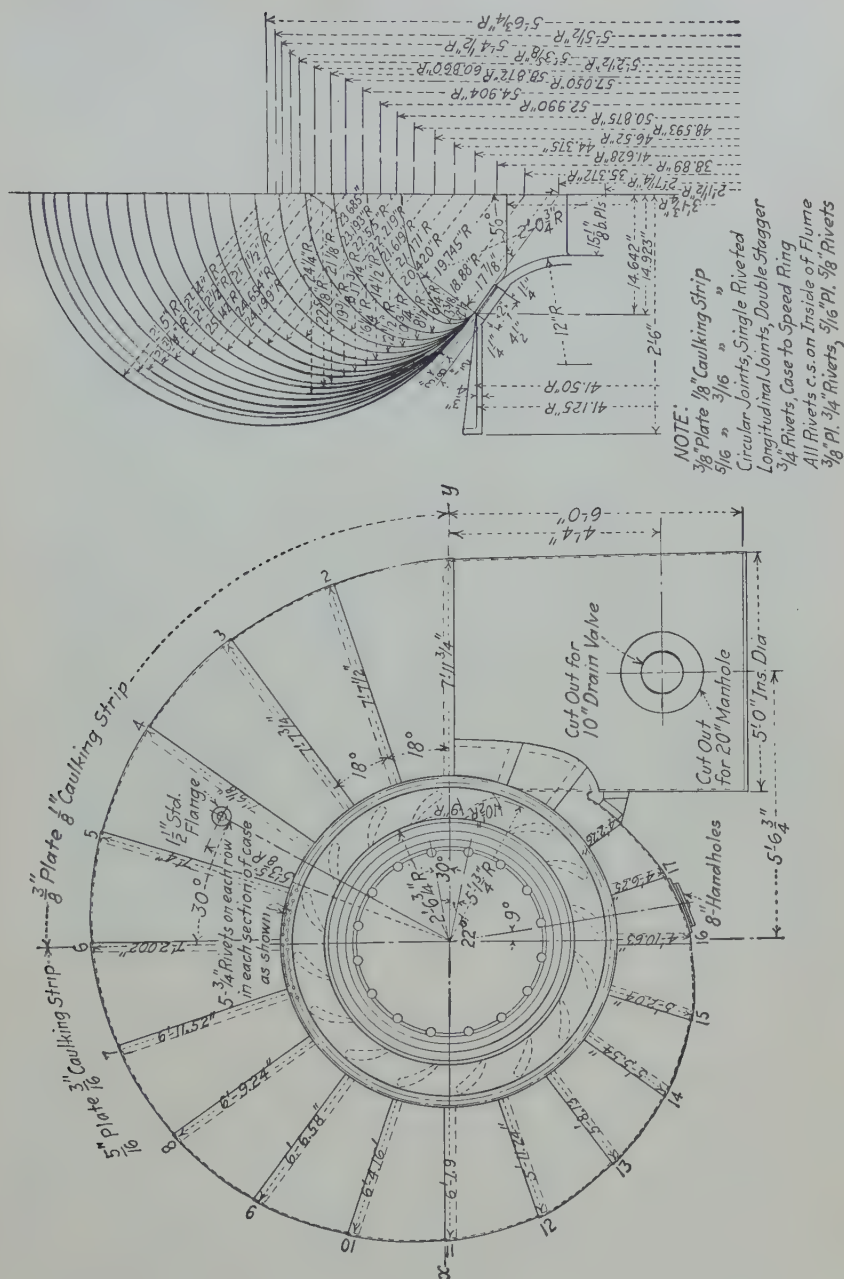


Fig. 171.—Details of plate steel scroll case—Estes-Rainbow Plant of J. & J. Rogers Co., Au Sable Chasm, N. Y.

of concrete in the power-house substructure. The cast-steel casing is relatively stiff and contributes to the stability of the substructure. Moreover, the latter can be shop assembled and tested under pressure, whereas the plate-steel casing can be tested only in place after all riveting is completed. As regards cost, taking into account erection as well as materials, there is no great difference between these two types.

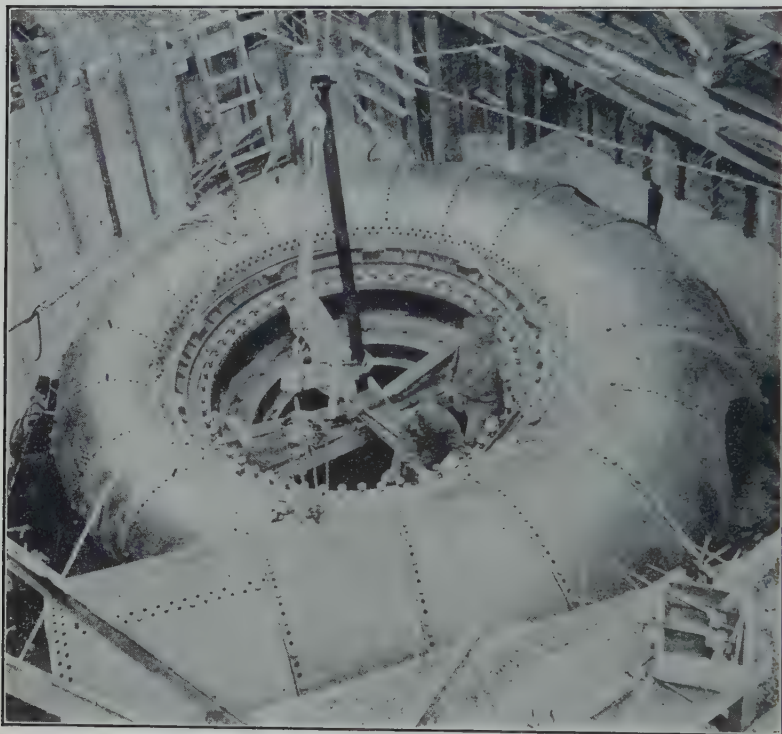


FIG. 172.—Assembling steel-plate spiral casing in the field—90-in. inlet diameter—Kern River Plant, San Joaquin Light and Power Corporation, Calif. (Courtesy of Allis-Chalmers Manufacturing Company.)

Formerly the cast-steel scroll casings were constructed in radial sections and bolted to the circumferential flanges of a separately cast speed ring. In a more recent and better procedure devised by H. B. Taylor, each radial section is split transversely (with staggered transverse flanges), and the speed-ring stay vanes are cast integrally with the small inner section (see Fig. 173). Other manufacturers in the case of large scrolls cast the speed-vane ribs integrally with the scroll casing and divide the casing into a large number of sections.

Scroll Casing for Horizontal Units.—As previously stated, where wheels are to have a relatively small discharge capacity under high heads, a horizontal setting may be desirable, chiefly for better accessibility.

In this case the penstock will normally approach the wheel from underneath on the level of the scroll entrance, thus requiring the wheel and generator shaft to be placed parallel to the length of the power house, as in the case of the Marlboro, N. H., plant (see Fig. 294, Chap. XIII). The wheel-generator unit may also be placed with shaft at right angles with the length of the power house by the arrangement shown in Fig. 174, where the penstock runs the length of the power house under the wheel units, with upward branches to each unit.

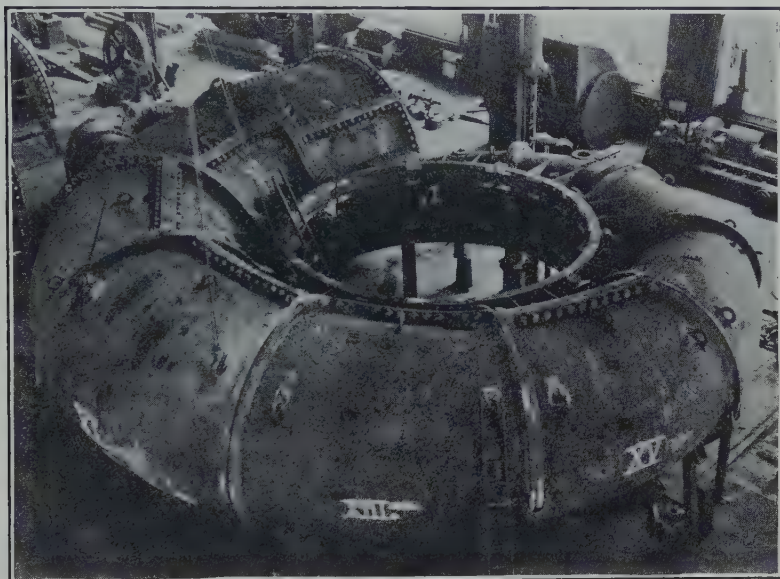


FIG. 173.—Sectional cast-steel spiral casing. Shop assembly for 70,000 h.p. wheel unit of Niagara Falls Power Company. (Courtesy I. P. Morris Department, William Cramp and Sons.)

Draft Tube.—The various common forms and arrangements of draft tube, as well as the theory involved in their design, were discussed in Chap. IV.

Referring to Fig. 87, Chap. IV, the use of the vertical flaring tube (a) with the open wheel pit requires the floor of the pit to be arched or reinforced to carry the load of wheel, generator, etc. Generally each unit will occupy a bay with concrete piers separating adjacent bays (see Fig. 162, page 396). The spacing of units, as already noted, will usually be fixed by allowable flume velocity above the wheel, and this spacing will commonly give a satisfactory tailrace velocity with a reasonable depth of the latter.

The concrete quarter-turn or elbow form of draft tube (Fig. 87b, Chap. IV) is simply an opening or channel in the power-house sub-

Draft-tube Dimensions.—At wheel discharge or the top of the draft tube, the cross section will be fixed by the wheel design and will always be circular. As shown in Fig. 88, Chap. IV, the velocity at the wheel-discharge point commonly varies from about $0.4\sqrt{2gh}$ for low heads to about $0.12\sqrt{2gh}$ for high heads. Similarly, at exit from draft tube, the velocity will be practically fixed, varying from about $0.10\sqrt{2gh}$ for low heads to $0.02\sqrt{2gh}$ for high heads. In determining the elevation of the top of the draft tube, there must be considered: (1) the necessary depth of the wheel below head-water level in the case of low-head developments; (2) the necessary height of wheel-gate pit and power-house basement or equipment above the tailrace flood level; (3) the requirements for length

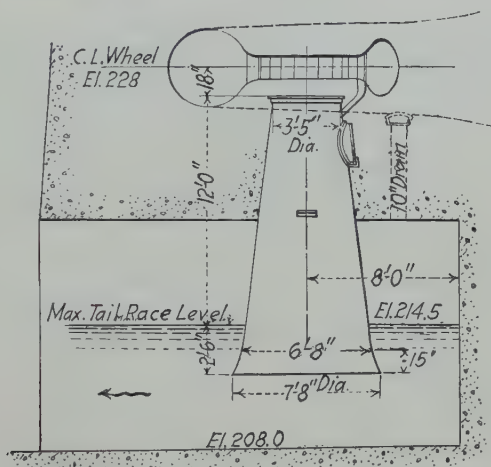


FIG. 175.—Vertical flaring steel draft tube—Estes-Rainbow Plant.

of draft tube, where, as with low-head plants and large discharge capacity, a considerable transition in area and shape of draft tube must be made between wheel and tailrace; (4) the necessary limitations as between height of draft tube and velocity at wheel discharge as discussed in Chap. IV, in general the former lessening as the latter increases.

The elevation of the end or mouth of the draft tube will be fixed for a given cross section by minimum tailrace water level, allowing a suitable seal or depth of water (usually 2 or 3 ft.) over the top end of the tube.

Vertical Flaring Draft Tube.—Where conditions permit, the vertical flaring draft tube gives good results. The dimensions of such a form of tube as used at the Estes-Rainbow plant, New York, for which the details of the spiral casing were shown in Fig. 171, are shown in Fig. 175. This tube is of $\frac{3}{8}$ -in. plate steel, 16 ft. long, measured from center of wheel, and in general has a flare of 3 in. to the foot, with a somewhat greater flare in the last 15 in. Tailrace excavation was in rock, with original surface varying from about an elevation of 218 to 212 ft. and excavated

to an elevation of 208 ft. Extreme flood level in the tailrace is at an elevation of about 226 ft. or near the top of the draft tube. Note the manhole in the side of the draft tube and also the small pipe drain which permits the space above the runner hub to be drained into the draft tube through the nose casing, for the purpose of reducing the hydraulic thrust on the runner. The 10-in. disk-type drain for the spiral casing also shows in Fig. 175.

Elbow Type of Draft Tube.—Where a vertical draft tube would require too much excavation for tailrace, particularly with wheels of large dis-

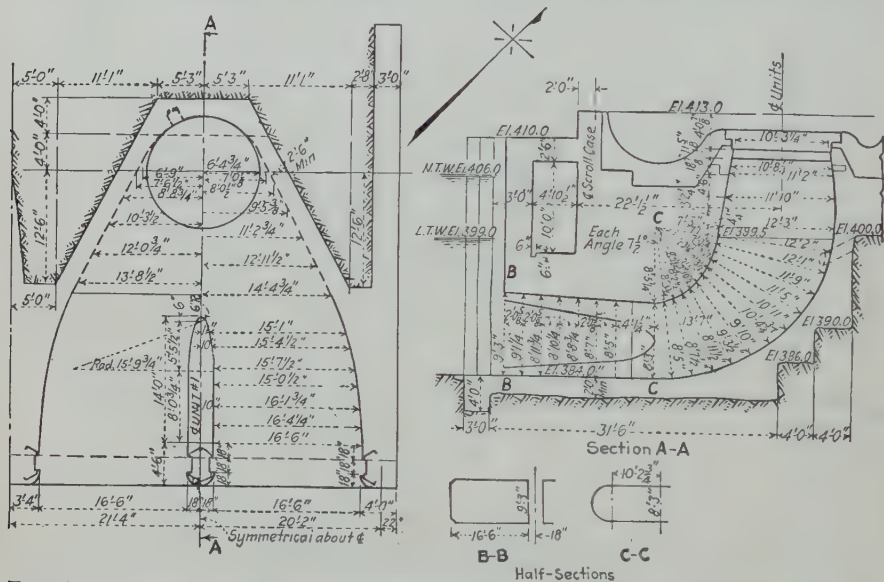


Fig. 176.—Details of draft tube—Bartlett's Ferry plant. (Courtesy of Stone and Webster Engineering Corporation.)

charge capacity, the 90-deg. turn or elbow type may be used. Further to save excavation with this type of tube, its cross section is gradually changed from circular at wheel discharge to approximately rectangular or elliptical at the mouth of the tube. A recent example of this kind of draft tube is shown in Fig. 176, as used at the Bartlett's Ferry, Georgia, plant of the Columbus Electric and Power Company (see also Fig. 302, Chap. XIII). These are large wheel units, using about 2000 sec.-ft. discharge and developing 22,000 hp. under a head of 112 ft., so that a central pier is required as the draft tube widens out. The cross section of the draft tube is circular for a short distance from the wheel, gradually merging into a rectangular section with rounded ends or sides. At its mouth the section consists of two rectangles, each 16 ft. 6 in. wide by 9 ft. 3 in. high, with 12-in. triangular fillets in the outside corners and separated by a 3-ft. pier. Areas and velocities are as follows:

Point in draft tube	Area, square feet	Velocity, feet per second	Ratio $\frac{v}{\sqrt{2gh}}$
Entrance.....	82.5	24.6	0.29
Mouth.....	304	6.6	0.08

The velocity at the mouth of the tube is unusually high (see Fig. 88, Chap. IV) and represents a velocity-head loss of about 0.7 ft. of head. This design was adopted as the most efficient after extensive tests upon

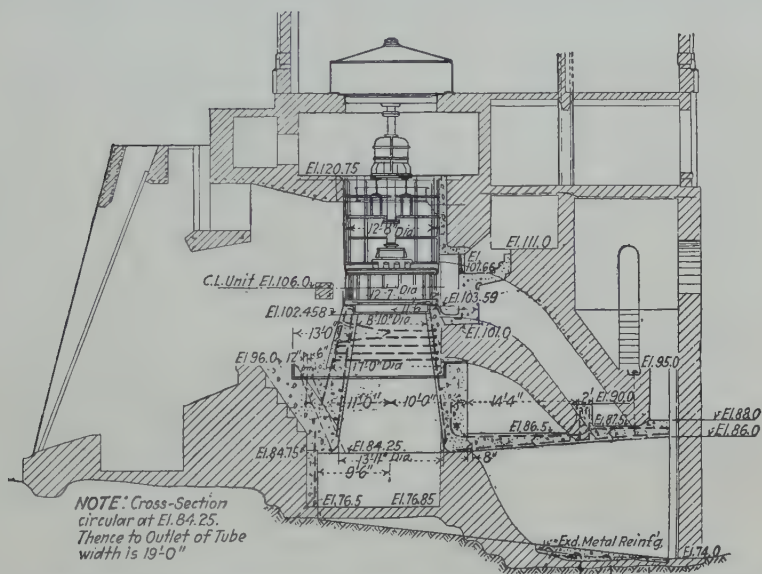


FIG. 177.—Reconstruction wheel units at Vernon plant—details of arrangement. (Courtesy of New England Power Company.)

model tubes at the laboratory of the wheel manufacturers (S. Morgan Smith Company) and should be carefully studied.

Another form of elbow type of draft tube, sometimes called the "stocking heel" from its shape, is illustrated by Fig. 177, being the tube for the reconstructed units at the Vernon station of the New England Power Company (see Chap. IV, Fig. 83). While the change in direction is rather abrupt, this appears to be a fairly efficient type of tube. Substantially the same arrangement is used for units 3 and 4 at the Mitchell dam of the Alabama Power Company.

Other Types of Draft Tube.—The hydracone draft tube is illustrated by the Davis Bridge wheel settings (see Fig. 291, Chap. XIII, and Fig. 165, page 399) and the Moody spreading tube by the Searsburg plant (see Fig. 293, Chap. XIII, and Fig. 170, page 410). These should be

analyzed from the design and construction viewpoints. They are efficient but generally more difficult and costly to construct than the types previously discussed. The hydraucone in its later form is, however, simplified in construction and, where the size of wheel permits the use of a steel tube, is not greatly different as regards construction from the vertical flaring tube. The central cone of the Moody spreading tube is difficult to construct and is possibly subject to wear and deterioration due to high velocities, the extent of which is a matter for further experience to determine.

Tailrace and Foundations. *Reaction-wheel Settings.*—The width of tailrace channel under the power house will depend upon the unit spacing and necessary thickness of piers or walls between the unit bays. The depth of tailrace will then depend upon the assumed velocity, which is in theory, as with penstock or canal, that which will give a minimum of annual cost (of tailrace and power lost in velocity head) and will, therefore, depend largely upon the cost of the tailrace channel. Usually a velocity of from 2 to 4 ft. per second will be suitable.

Where the power house is close to the river, the tailrace may be the river itself; in other cases a tailrace channel of some length may connect with the river. The cost of excavation from wheel pit to river and the cost of masonry walls for the bays of the power-house substructure are the chief items to be considered in determining the proper depth of the tailrace channel, keeping in mind that in the case of the elbow type, the Moody spreading tubes, and some forms of hydraucone, where there is no open channel under the power house, the substructure becomes practically a mass of concrete with water passages, rather than bays between piers.

Foundation Requirements.—The problem of a suitable foundation for the power house depends on the foundation material. Where ledge is

SAFE-BEARING CAPACITY OF SOILS—SHORT TONS PER SQUARE FOOT

Kind of material	Minimum	Maximum
Rock:		
Hardest, in native bed.....	200	
Equal to best ashlar masonry.....	25	30
Equal to best brick masonry.....	15	20
Equal to poor brick masonry.....	5	10
Clay:		
In thick beds, dry.....	6	8
In thick beds, moderately dry.....	4	6
Soft (protected against lateral displacement)....	1	2
Gravel and coarse sand, well cemented.....	8	10
Sand, dry, compact, well cemented (coarse grained).	4	6
Sand, clean, dry (fine grained).....	2	4
Quicksand, alluvial soils, etc.....	0.5	1

available within moderate depth, the foundation should always be carried to it, and in this case pressures of 20 to 25 tons per square foot, or, with hard sound rock, up to the safe working strength of the concrete can be safely used, and the problem of design is simple. With an earth foundation where the soil capacity is definitely limited, the whole base must sometimes be made to bear the weight, in which case the floor of the tailraces, as well as the walls, must be designed to distribute the load properly over the foundation material. With an earth foundation its bearing capacity depends upon its composition, the extent to which it is confined, and its moisture content. Hool and Johnson¹ give in the table on page 418, the safe loads for soils of different kinds.

Foundation Loads.—The foundation (with vertical wheel units) must carry the following permanent loads:

1. Generator:
 - a. Rotor.
 - b. Stator or frame.
2. Wheel:
 - a. Scroll case (filled with water).
 - b. Pit liner.
 - c. Speed ring.
 - d. Runner.
 - e. Runner thrust.
 - f. Draft tube (where of steel).
3. Dead load:
 - a. Foundation.
 - b. Floor (including generator pedestals).
 - c. Superstructure.
4. Live load on station floor and other floors.

In the initial computations, the dead load of foundation, floor, and superstructure can only be approximated. The live floor load may usually be taken as the weight of the heaviest single piece of machinery, as generator, rotor or stator, etc., placed anywhere there is room for it on the power-house floor. The wind or snow load from the roof will also be transmitted through the superstructure to the foundation.

Arrangement of Foundations.—Where the draft tube is molded in the concrete of the substructure, as with elbow-type draft tubes, etc., the problem of design becomes one of providing sufficient strength where loads are concentrated—as at the pit liner and speed ring—using reinforcement or other steel as necessary. The reinforcement of a scroll case and penstock or flume is a very complicated matter, as the internal water pressure with the proper allowance for added pressure due to quick closure of the turbine gates produces very heavy internal pressures which may and usually do exceed greatly the superimposed weight.

¹ "Concrete Engineers' Handbook," p. 583, 1918.

The use of moderate stresses in steel for resisting these forces of pure tension has been discussed under open-flume settings and is employed here, as unless low stresses are used, the concrete strength in tension will be exceeded, cracks will be formed, and troublesome leakage occur. The walls, roof, and floor of the wall should be designed as a monolithic structure and all cases of loading investigated which may affect the reinforcement or size and strength of the structure.

A number of difficult structural problems which arise in a structure of this kind may be settled in advance by conference with the manufacturer. For example, the speed ring and pit liner should be constructed so as to carry the reaction from the top of the scroll case directly down to the foundations, and in the case where a tension is developed under maximum surge conditions the connection of the pit liner to the concrete should be amply sufficient to insure the transfer of the stress and provide space enough for proper reinforcing.

It should be borne in mind that water pressure is always being exerted, and that, unlike most structures, the members are usually stressed at or near to the limit of design at all times and conservative stresses in both steel and concrete are advisable, particularly when the exact analysis of stresses in arch structures is impossible.

Where a vertical steel draft tube is used with tail-pit bays, the roof of the latter may be of either arched or slab construction. This will concentrate the loads at the piers between bays, which must have footings suitable for the material upon which they rest. If arches are used, the end walls of the substructure will have to carry an unbalanced thrust and must be widened out accordingly or reinforced. If slab construction is used for the tail-pit roof, this must be properly reinforced with either steel rods or a structural steel framework designed especially to carry the loads coming through the speed ring, which are practically the loads previously listed, for one bay. A design of the latter kind is shown in Fig. 178, as used at the plant of the Utilities Power Company, Bristol, N. H.¹

Construction Joints.—Certain construction joints must be fixed as a part of the design, consistent with the practical placing of concrete and the different parts of the machinery. Thus, in the case of the Turners Falls plant (see Fig. 298, Chap. XIII) such a joint was planned near the top level of the draft tube (which is of the elbow type) and concrete first carried to this level. After placing the speed ring and pit liner of the wheel, concrete was again placed up to floor level.

In Fig. 178 such a construction joint was used at the level of the top of the steel girders adjacent to the draft tube, the latter being concreted into position with the initial pouring. The entire wheel casing, including

¹ HENDERSON, W. D., "Structural Design Features of a Hydroelectric Development," *Jour. Boston Soc. Civil Eng.*, April, 1925.

speed ring, pit liner, and scroll case, as well as the taper portion of the penstock line, were then placed before any further concrete was poured.

Generator Pedestal.—Where a concrete pedestal is used for the generator base, as in the single-floor type of station, it must be provided with doorway for access to the wheel-gate mechanism, and often other openings, for ventilation purposes, belts for governor, governor pumps,

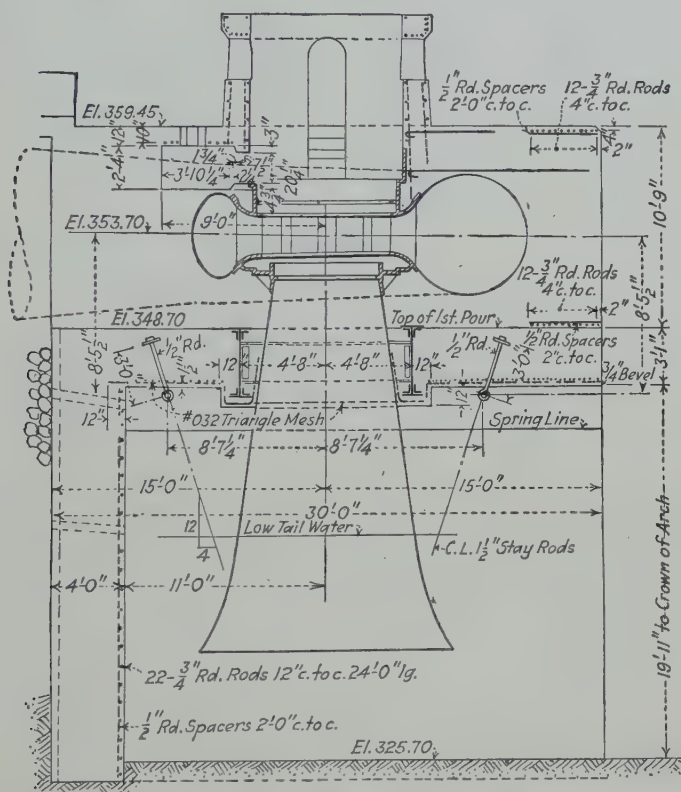


FIG. 178.—Utilities Power Company, Bristol, N. H. Wheel and generator setting—sectional detail.

exciter, etc. Some details of such a pedestal are shown in Fig. 178. In this case in addition to the vertical loads coming to the pedestal an impact allowance of 50 per cent of the weight of generator, wheel runner, and the runner thrust was assumed; also a horizontal impact allowance was assumed to be applied in one direction at the top of the generator equivalent to 50 per cent of its total weight, representing possible dynamic unbalance which might occur due to short-circuit conditions.

Foundation Bolts.—Foundation bolts for wheel, generator, etc., should usually be set in the concrete in pipes, thus permitting some lateral movement when the machinery is erected, followed finally by grouting. With large bolts it may be desirable to leave a recess or pocket in the concrete large enough to contain the bolt, end washer, etc., to be filled later with concrete.

Impulse-wheel Settings.—The tailrace of the impulse wheel is commonly an approximately rectangular passage, running from a point under the wheel to a point outside the power-house foundations, where it enters the tailrace channel or the river. Due to the smaller discharge of the impulse wheel, as well as higher allowable velocity, the tailrace passage is much smaller than that of the reaction wheel. A common arrangement is shown in Figs. 304 and 305, Chap. XIII, for the Mystic Lake, Montana, development, where the bottom of the tailrace is only 12 ft. below power-house floor level and is but 6 ft. wide.

The loads of wheel and generator are applied more directly upon the foundations, however, in the case of impulse-wheel settings, being concentrated at the two bearing points for either double- or single-overhung units.

Where an auxiliary nozzle is used for speed regulation, arrangement must be made to receive the jet which is wasted for short intervals from time to time. Figure 68, Chap. IV, shows such an arrangement of baffle plate or deflector, backed up with steel beams and set in the foundation masonry.

The nozzle and its accessories, penstock gate, penstock approach, etc., are placed in the basement just upstream from the wheel.

Floor and Basement Arrangement.—The general tendency is now toward the omission of any general basement for the power house, but provision is made for any equipment below floor level in chambers, often made more accessible by a stairway from the main floor and connecting passages. Space may be required here for oil pumps, gates, etc., but the governor and its auxiliaries are now generally placed on the main floor. In some cases the leads or cables from the generator are laid under the floor, carried on supports in passageways to and from the switchboard, or they may be laid in metal or fiber ducts placed in the concrete floor. Usually ventilating ducts are also constructed in the substructure, so arranged that air may be taken from outdoors or (in cold weather) from the inside of the station.

The use of the single-floor type of station, with generator set upon a concrete pedestal (as illustrated by the Davis Bridge plant (Fig. 291, Chap. XIII), permits ready access to the wheel-gate mechanism and allows placing the wheel and generator auxiliaries on the main floor, all in sight from above.

POWER-HOUSE SUPERSTRUCTURE

General Arrangement.—The general arrangement of floor and building has already been discussed under substructure, as the former must be planned in a general way prior to the design of the latter. The generating room, the main portion of the power house, contains the main units and their accessories, and usually there is a power- or hand-operated overhead crane which spans the width of the power house. The switch-board and operating stand are usually near the middle of the station, either at floor level or, for better visibility, on the second floor or at a level above the main floor. Usually an auxiliary bay or section of the power house will be required upstream from the main units for the switches, bus connections, and outgoing lines. If transformers are located inside the station, these will also be in the auxiliary bay, commonly at floor level and shut off from the main floor by steel doors or shutters. The plans and details of the various stations shown in Chap. XIII should be studied with reference to general arrangement.

Floors.—The main floor, of concrete, may have a granolithic finish or, better, be of tile, selected to harmonize with the wall finish. Other floors will generally be of reinforced concrete, with smooth finish, designed in accordance with the equipment to be carried.

Frame and Walls.—The building frame will usually be of structural-steel columns spaced to accord with the spacing of the units and directly carrying the crane runway beams, as well as the roof trusses. Reinforced concrete is also often used for the walls and crane supports.

For small stations, the walls, of brick or concrete, may be used to support the roof, the crane runway beam being carried on pilasters, which are a part of the masonry walls.

Where steel-frame construction is used, a thickness of 12 in. is usually adequate in the walls. Where the walls carry the roof and crane loads, their thickness must be proportioned accordingly.

The finish of the interior walls may be of pressed brick or, if of concrete, painted; in either case preferably of light-colored tint for better lighting effect.

The windows are an important detail and should usually constitute a large proportion of the area of the walls of the power house. On the long side of the house, their width and arrangement should accord with the unit spacing, and they will usually extend up to or above the crane level. Typical construction of power house superstructure with brick walls is shown in Fig. 179, for the station 2 of the Holyoke Water Power Company and in Fig. 180 for the Crescent development of the State of New York on Mohawk River. A station with concrete walls is shown in Fig. 181 for the plant of the Lockwood Company at Waterville, Me. (for details of these power houses see Table 104, Chap. XII).

In Fig. 182 is shown a general view of the Lower Fifteen Mile Falls (Comerford) plant on the upper Connecticut River. The power-house superstructure is finished outside with buff-colored brick and is of a general design to harmonize with its setting just below a massive 170-ft. concrete dam. A flat roof with low parapet is used and projecting buttresses on the downstream side, symmetrically spaced with the four generating units (see Table 104, Chap. XII).

In the first two of the above power houses the windows extend without break to near the crane level, with brick arched walls above each window at Holyoke and lintel construction at Crescent. In the Waterville plant



Fig. 179.—Power-house exterior—Holyoke Water Power Company, No. 2 station. (Courtesy of Holyoke Water Power Company.)

there are two horizontal rows of main windows (with separating wall sections) which reach to the crane level, above which are curved windows.

In the Comerford power house there are two main windows between each pair of buttresses (only one of the two is visible in Fig. 182). The rectangular openings with shutters, below the windows, are air inlets for the generators, and the small shuttered openings near the top of the buttresses are for generator air outlets. The Pit River 3 power house (Fig. 183) is an attractive recent design of a concrete superstructure.

Window sashes should be of metal (as in Figs. 179 and 180) and the glass usually with wire-netting reinforcement. Often the upper portions of the windows are made of ribbed or non-transparent glass to keep out direct sunlight. Sections of the windows should be arranged to tilt open, for ventilating purposes.

The main door is usually at one end of the station and often arranged of sufficient size to permit a railroad car with machinery, etc., being brought in upon a track on the floor. A type of rolling lift door is well

adapted for the large or main door. The service door or doors will be located according to circumstances. Often a service door is installed as a part of the main door.



FIG. 180.—Power-house exterior—Crescent station, Mohawk River, N. Y. (*Courtesy of State Engineer, N. Y.*)

Roof.—The common type of roof is illustrated in Figs. 179, 180, and 181; it has a relatively flat slope (a pitch of from $\frac{1}{4}$ to $\frac{1}{2}$ in. per foot) with tar and gravel finish on wood or concrete base, or with concrete-top-finish waterproofing and concrete base. There is usually a coping projecting above the roof level and harmonizing with the wall construction.



FIG. 181.—Power-house exterior—Lockwood Company, Waterville, Me.

Where roof trusses are required, they are usually constructed of steel angles with riveted-plate connections, the trusses having usually relatively shallow depth. The roof should be designed to carry a dead load

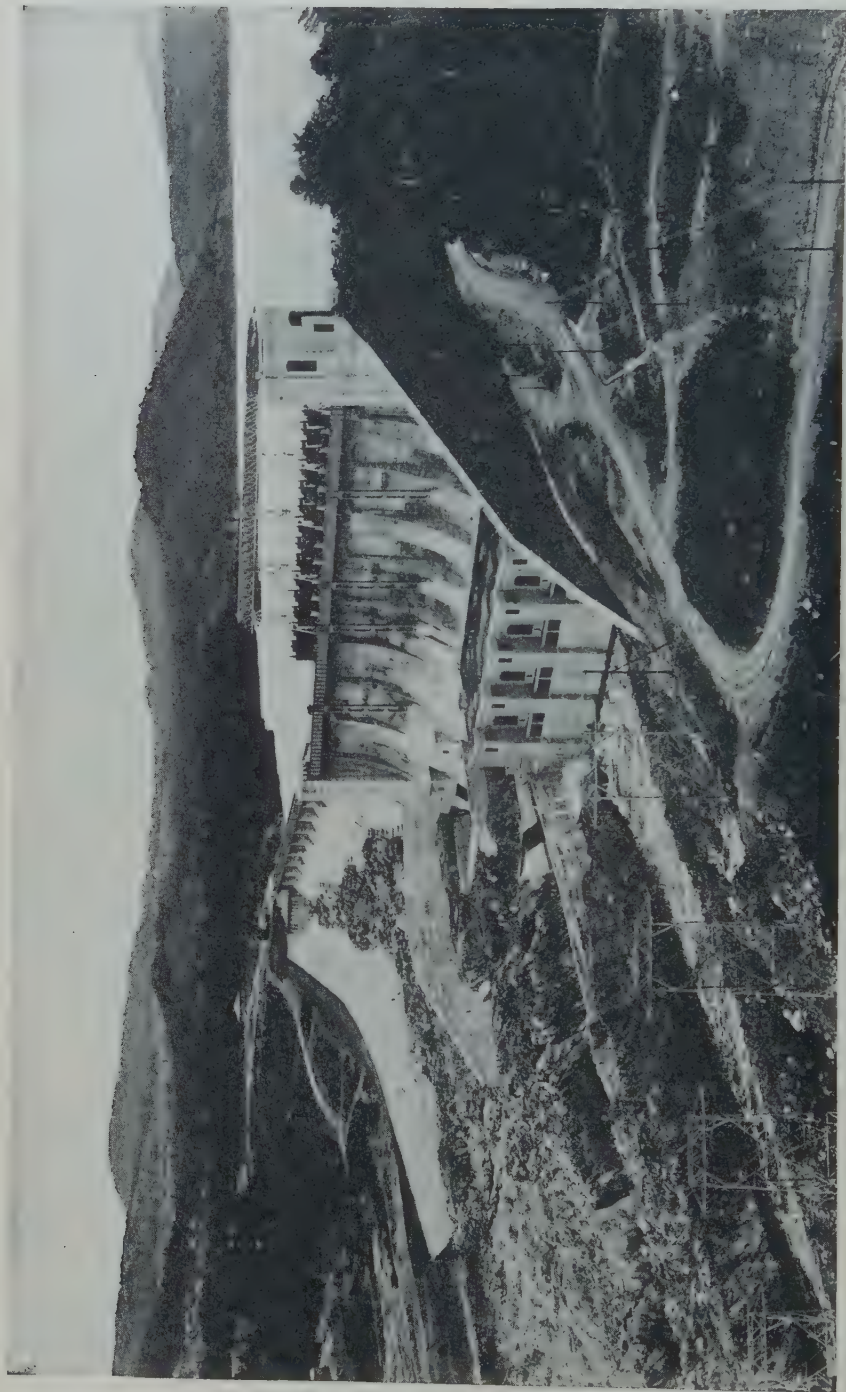


Fig. 182.—Lower Fifteen Mile Falls (Comerford) Development. (Courtesy of New England Power Association.)

consistent with its type of construction and a snow load varying with the latitude of from 10 to 40 lb. per square foot.

A sloping roof (1 on 2 or 3) is also used, with metal, slate, or tile exterior finish (see Figs. 304 and 305, Chap. XIII), requiring triangular roof trusses which must be capable of withstanding the normal component of a wind load of 30 lb. per square foot. This type of roof is often arranged with a monitor and tilting windows which can be operated from the station floor, for ventilation purposes.



FIG. 183.—Pit River No. 3 Development—Pacific Gas and Electric Company—Power House.

The roof trusses, except for small stations, should be supported by and rigidly connected to the wall columns, which also by a suitable connection carry the crane runway girders. The latter are usually supported directly by the wall columns, which are made deep enough for this purpose, as with the Crescent power house (Fig. 299, Chap. XIII), or they may be carried by some form of knee-brace connection.

Crane and Crane Girders.—A traveling crane is an important part of the power-house equipment. As previously stated, it will span the width of the house and be arranged to travel for its entire length. In fixing the elevation of the crane rail above the floor, it is essential that sufficient headroom be provided for lifting and carrying along any of the various machine parts. There is always a shaft coupling between generator rotor and wheel runner, and provisions should be made so that

either one of these may be lifted out without dismantling the stator and carried the length of the station. This requirement of headroom is shown in Table 75 in line 42, by the distance from generator base to top of crane rail, which varies usually from about 25 to 40 ft., depending upon the size of units. Note that the generator base, in the case of the single-floor type of station, is considerably above the floor level.

The distance from crane rail to bottom of roof truss (line 43, Table 75) varies, practically with the crane capacity, from about 5 to 12 ft.

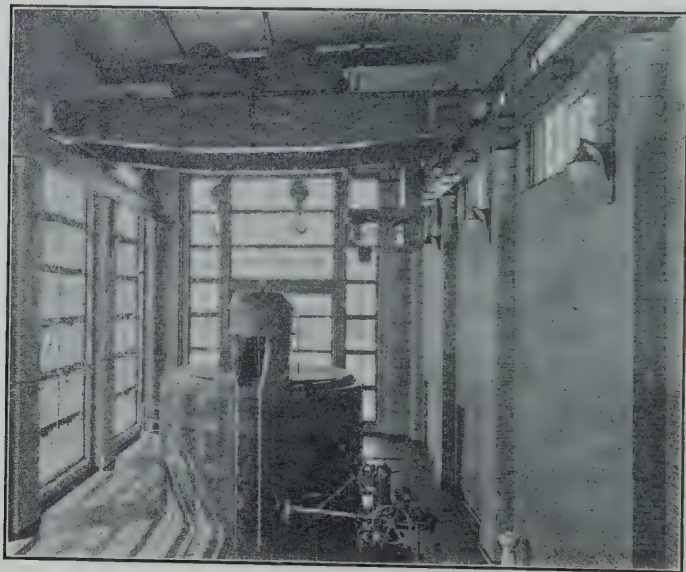


FIG. 184.—Power-house interior and crane arrangement—Green Island Plant, Henry Ford and Son. (Courtesy of Cleveland Crane and Engineering Company.)

The usual height of station, from generator base to bottom of roof truss, will, therefore, be between 30 and 50 ft.

In Table 77 are given dimensional data for one make of crane, and in Fig. 184 one of these cranes is shown at the Green Island plant on the Hudson River. In this case the crane runway girders are supported on brick pilasters, the span between crane rails being 29 ft. 9½ in. This is a double-drum crane with two hoists of 20 tons each. The interior finish, roof detail, window arrangements, lighting facilities, etc., of this station are also of interest.

The desirable capacity of crane will depend upon the size of units. Usually it should be capable of lifting the generator rotor, shaft, and wheel runner as a whole for a short distance for blocking up upon the generator-field rim so that the thrust bearing may be inspected or repaired. In line 44, Table 75, are given crane capacities for a wide range of station conditions, which vary from 20 to 300 tons. In the

case of the very large Cedar Rapids station, two 150-ton cranes, 20 ft. apart, are used, with lifting beam provided for making a 300-ton lift. Keokuk, another large station, has one 150-ton crane.

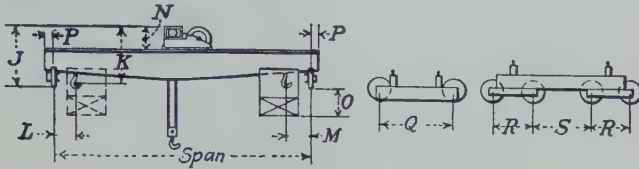


TABLE 77.—STANDARD CRANE CLEARANCES
(The Cleveland Crane and Engineering Company)

Rail, pounds as per yd.	Span, feet	J	K	L	M	N	O	P, inches	Q	R	S	Wheel load, pounds	Rail, pounds per yd.
5	25	5' 0"	7' 6"	2' 7"	3' 3"	3' 0"	7' 9"	7	9' 6"			11,100	60
5	50	5' 6"	7' 8"	2' 7"	3' 3"	3' 0"	7' 9"	8	9' 6"			14,200	60
5	80	6' 0"	8' 0"	2' 7"	3' 3"	3' 0"	7' 6"	9	10' 6"			19,900	60
5	100	6' 3"	7' 8"	2' 7"	3' 3"	3' 0"	7' 6"	9	12' 6"			25,000	70
10	25	5' 2"	7' 8"	2' 7"	3' 3"	3' 3"	7' 9"	8	9' 6"			16,100	60
10	50	6' 0"	8' 5"	2' 7"	3' 3"	3' 3"	7' 6"	9	10' 6"			20,600	60
10	75	6' 3"	8' 9"	2' 7"	3' 3"	3' 3"	7' 6"	9	11' 6"			27,000	70
10	100	6' 8"	9' 2"	2' 7"	3' 3"	3' 3"	7' 3"	10	11' 6"			34,300	70
15	30	6' 7"	10' 2"	3' 6"	4' 6"	3' 9"	7' 9"	8	10' 6"			24,000	60
15	60	7' 0"	10' 2"	3' 6"	4' 6"	3' 9"	7' 6"	9	11' 6"			30,200	70
15	75	7' 4"	10' 2"	3' 6"	4' 6"	3' 9"	7' 3"	10	11' 6"			34,500	80
15	100	7' 11"	10' 3"	3' 6"	4' 6"	3' 9"	7' 3"	10	12' 6"			42,800	80
20	25	6' 10"	10' 2"	3' 6"	4' 6"	3' 9"	7' 9"	9	10' 6"			27,200	70
20	50	7' 0"	10' 2"	3' 6"	4' 6"	3' 9"	7' 6"	10	11' 6"			32,900	70
20	75	7' 6"	10' 4"	3' 6"	4' 6"	3' 9"	7' 3"	10	11' 6"			40,900	80
20	100	8' 0"	10' 7"	3' 6"	4' 6"	3' 9"	7' 3"	10	12' 6"			50,000	80
30	20	7' 0"	11' 0"	4' 3"	6' 3"	4' 3"	8' 0"	10	12' 6"			36,000	80
30	60	7' 3"	10' 6"	4' 3"	6' 3"	4' 3"	8' 0"	10	12' 6"			48,000	80
30	80	8' 6"	10' 6"	5' 0"	6' 3"	4' 3"	8' 0"	12	14' 6"			61,500	100
30	100	9' 9"	11' 9"	5' 0"	7' 6"	4' 3"	8' 0"	12		5' 3"	4' 9"	52,500	100
50	40	7' 8"	11' 0"	4' 6"	6' 3"	4' 3"	8' 0"	12	13' 0"			65,000	100
50	60	7' 10"	11' 0"	4' 6"	6' 3"	4' 3"	8' 0"	12	14' 6"			72,900	100
50	80	8' 10"	10' 9"	4' 6"	7' 0"	4' 3"	8' 0"	12		5' 3"	4' 9"	47,700	100
75	40	8' 10"	12' 6"	4' 6"	6' 6"	4' 8"	8' 0"	12		5' 3"	5' 3"	50,900	100
75	60	8' 10"	12' 6"	4' 6"	7' 0"	4' 8"	8' 0"	12		5' 3"	4' 3"	54,200	100
75	80	9' 6"	12' 0"	4' 6"	7' 0"	4' 8"	8' 0"	14		5' 3"	6' 9"	65,000	100
100	40	8' 10"	13' 0"	4' 6"	7' 0"	4' 8"	8' 0"	14		5' 3"	10' 3"	63,400	100
100	50	9' 0"	13' 6"	4' 6"	7' 6"	4' 8"	8' 0"	14		5' 3"	6' 9"	66,300	100
100	65	8' 10"	13' 0"	5' 0"	7' 6"	4' 8"	8' 0"	14		5' 3"	6' 9"	73,300	100

Dimensions Q and S and wheel load will change depending on height of hook travel which changes spread of trolley.

Above dimensions take care of from 25' 0" to 35' 0" hook travel and are for standard cranes but not for steel-mill cranes and subject to change due specifications.

The crane runway girders must be designed to carry:

1. One-half the weight of the bridge or crane frame.
2. The reaction from the crane trolley and its live load when nearest to the runway girder.
3. The weight of the bridge truck.

These loads will be applied to the runway girder at two points distant from each other the span of the bridge-truck wheels. The maximum moment for this loading will occur, as may be shown, when one of the wheel loads is at a distance of $b/4$ from the middle point of the girder where b is the span of the bridge-truck wheels, this moment being for the point at which the above-mentioned wheel load is placed.

Costs.—A detailed study of power-house areas, volumes, and costs is given in Tables 104 and 105, Chap. XII, followed by equipment-cost data, which will be useful in preliminary estimates.

Additional approximate costs of detailed items based upon a 200 per cent cost index (Table 122, Chap. XII) follow.

APPROXIMATE COSTS—POWER-HOUSE ITEMS

Concrete, plain (substructure), in place.....	\$15 per cubic yard
Concrete, reinforced (substructure), in place.....	\$20 per cubic yard
Concrete, reinforced (superstructure and floor slabs), in place.....	\$25-\$35 per cubic yard
Brick (superstructure), in place.....	\$100-\$125 per thousand
Precast stone (trim).....	\$4-\$5 per cubic foot
Structural steel, erected and painted.....	4-5 cts. per pound
Steel sash windows, glazed, in place.....	\$2.50 per square foot
Tile flooring, in place.....	\$0.70 per square foot
Granolithic floor, placed.....	\$0.70 per square foot
Gypsteel slab roofing, placed.....	\$0.30 per square foot
Roof waterproofing.....	\$0.30-\$0.50 per square foot

The cost of motor-operated power-house cranes may be estimated upon a ton-foot (of span) basis at \$4 to \$6 per ton-foot (the lower figure for very large cranes).

CHAPTER VIII

POWER HOUSE—ELECTRICAL

ELECTRICAL EQUIPMENT

Fundamentals.—Voltage, frequency, and other electrical principles, definitions, and characteristics are explained in some detail in Chap. XI, under Transmission Lines (pages 553–563).

GENERATORS

Classification.—The essential features of an electrical generator are (1) a field or assembly of magnets arranged to produce a magnetic flux, and (2) an armature or assembly of electric conductors arranged across the path of the magnetic flux. The field and armature are so mounted that by the application of mechanical force a relative motion is produced between the magnetic flux and the electric conductors inducing in the conductors an electromotive force. Since the field poles are arranged alternately positive and negative around the periphery of the generator, the polarity of the electromotive force induced in the armature is alternating.

The alternating-current synchronous generator or alternator delivers its induced alternating current directly to the external circuit and is used in hydroelectric stations where the output is to be transmitted over long lines, since the alternating current can be transformed up to suitable transmission voltage. It is also generally used where the output is to be distributed locally, as power and lighting service is usually alternating current.

The direct-current generator is provided with a commutator which rectifies its induced alternating current and delivers direct current to the external circuit. Direct-current generators are occasionally used where the output is to feed local railway or industrial loads which require direct current. Small direct-current generators called exciters are required in all power stations for energizing the magnetic fields of the alternators.

An induction generator is similar, physically, to an induction motor with a squirrel-cage secondary winding. Instead of revolving at slightly less than synchronous speed, as an induction motor does when consuming electrical energy, the induction generator is driven slightly above synchronous speed and, as a result, delivers electrical energy. In common with the induction motor, however, the induction generator consumes a

lagging exciting current, which in most cases is detrimental to the system but in some cases can be used to advantage partially to offset the leading exciting current consumed by the transmission lines. Advantages of the induction generator as compared with the synchronous generator include less short-circuit current, more rugged rotor construction, less danger of overvoltage resulting from overspeed, and less danger of overload. Disadvantages include smaller air gap, and lagging exciting current as mentioned above.

The single-phase alternator has one armature winding arranged to deliver a single-phase alternating current to a single-phase two-wire system. Single-phase generators are only occasionally used to supply special loads such as single-phase railways, electrochemical and electro-thermal industries, and in smaller sizes for lighting loads. The arrangement of the single-phase cores and windings is such that a single-phase generator is about 30 per cent larger than a two- or three-phase generator of equal capacity.

The two-phase alternator has two armature windings arranged to deliver two single-phase alternating currents, 90 electrical degrees apart in phase, to a two-phase four-wire system; hence, it is sometimes called a quarter-phase generator. If the center points of the two windings are connected, the generator is said to be interconnected. The point of interconnection may be used as a grounding point or as a neutral for a two-phase five-wire system if desired. In case the machine is not interconnected, two of the terminals, one from each winding, may be connected so that the machine can be used on a two-phase three-wire system, with common connection grounded if desired. Two-phase generators are frequently used in territories where two-phase systems are well established, but, due to the better transmission economy, it is often preferable to use three-phase generators and transmission line, although phase converters may be necessary at the load end.

The three-phase alternator has three armature windings arranged to deliver three single-phase alternating currents, 120 electrical degrees apart in phase, to a three-phase three-wire system. The delta-connected three-phase generator has the end of each winding connected to the beginning of the next winding, the three points of connection being used as line terminals. The star- or Y-connected machine has one end of each winding connected together in common, the three other ends being used as line terminals. The common connection may be used as a grounding point or as a neutral for a three-phase four-wire system if desired. The delta connection is sometimes used for low-voltage, high-amperage machines, but usually the star winding is preferred, because of the higher voltage between terminals as compared with the voltage of each winding and because of the better opportunity to apply relay protection. Three-phase generators are always preferred over two-phase or single-

phase machines except where the latter are needed to meet the local conditions mentioned above.

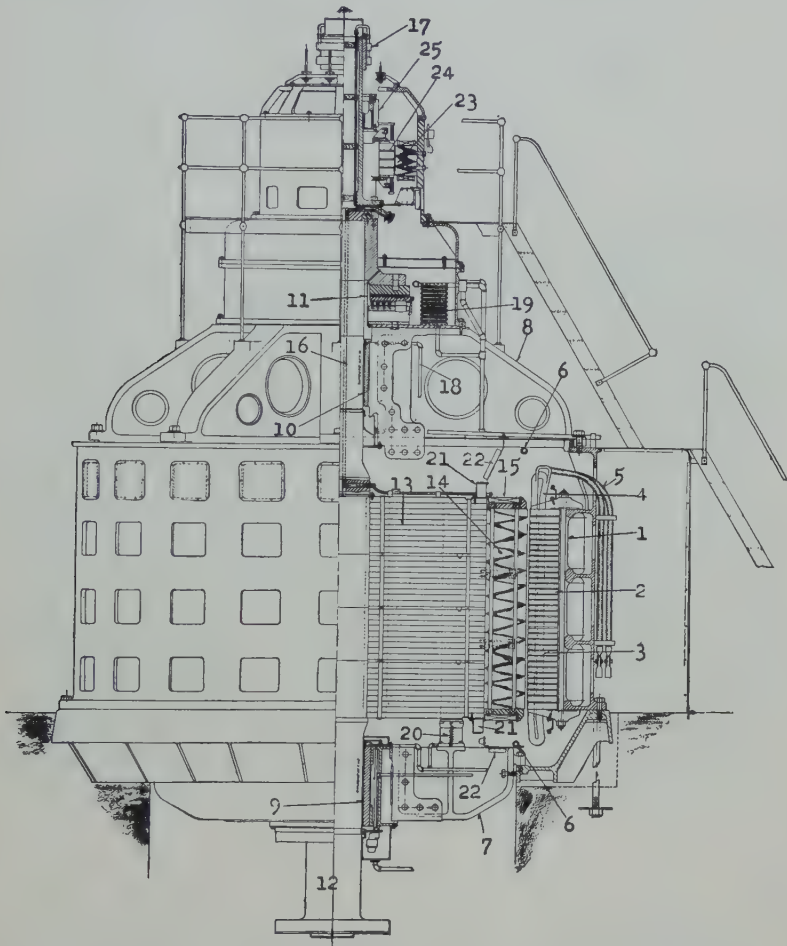


FIG. 185.—Vertical water-wheel-driven alternator—General Electric Company.

- | | |
|------------------------------------|--|
| 1. Stationary armature frame. | 14. Laminated-field pole. |
| 2. Laminated armature core. | 15. Field winding. |
| 3. Armature winding. | 16. Duct through shaft for field leads. |
| 4. Armature end connections. | 17. Collector rings. |
| 5. Armature leads. | 18. Oil piping for lubricating bearings. |
| 6. Fire-extinguisher water piping. | 19. Water piping for cooling bearings. |
| 7. Lower bearing bracket. | 20. Brake and jack. |
| 8. Upper bearing bracket. | 21. Fans. |
| 9. Lower bearing. | 22. Air baffles. |
| 10. Upper bearing. | 23. Exciter field. |
| 11. Thrust bearing. | 24. Exciter armature. |
| 12. Shaft. | 25. Exciter commutator. |
| 13. Revolving-field frame. | |

Construction.—A cross-sectional view through the axis of a typical vertical revolving-field type of moderate-speed water-wheel-driven alternator is shown in Fig. 185.

The rotating field or rotor may be one of two general types: (1) the distributed-pole type in which the windings are embedded in slots in the face of the cylindrical wheel so that the pole pieces are integral with the wheel and (2) the salient-pole type in which the windings surround individual pole pieces mounted on the periphery of the wheel. The former type is used for the highest speed generators on account of its greater strength and compactness, but the majority of water-wheel generators have salient-pole-type fields, built up of steel plates or cast-steel spiders with spokes. Field poles are laminated to prevent power

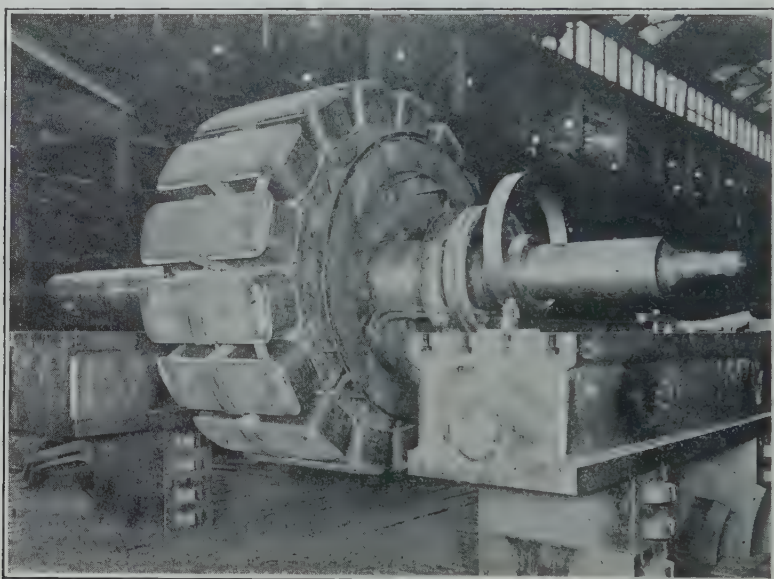


FIG. 186.—Generator rotor—Westinghouse Electric and Manufacturing Company.

losses by hysteresis and eddy currents. The coils are form wound and slipped over the poles before assembly. Cast-iron slip rings and carbon brushes are used to conduct the field current to the coils on the rotor. The brushes are held in contact with the rings by means of springs and are usually staggered to prevent wearing grooves in the rings. As water wheels are subject to runaway speeds of from 150 to 200 per cent of normal speed, it is necessary that water-wheel-driven generator rotors be designed to withstand safely the mechanical stresses due to approximately double normal speed.

Figure 186 shows a typical, moderately high-speed rotor with steel-plate wheel, salient poles, steel-plate fan, field leads, slip rings, journals, and one of the oil rings.

The stationary armature or stator consists of a circular steel frame supporting the core and coils. The core, like the field poles, is laminated to prevent losses. The coils are form wound and laid in slots in the core.

Figure 187 shows a typical half-stator frame and armature winding.

Bearings for horizontal generators are usually two in number, one at each end of the rotor. When the water wheel is overhung, the adjacent generator bearing must be designed to carry the weight and thrust of the wheel as well as its portion of the generator rotor. The bearings are generally of the babbitted, ring-oiled type (see Fig. 186).

Bearings for vertical generators generally consist of a thrust bearing at the top and two guide bearings, one above the rotor and one below, as shown in Fig. 185. When the rotor is closely coupled to the water wheel which has its own bearings, the lower generator bearing is sometimes

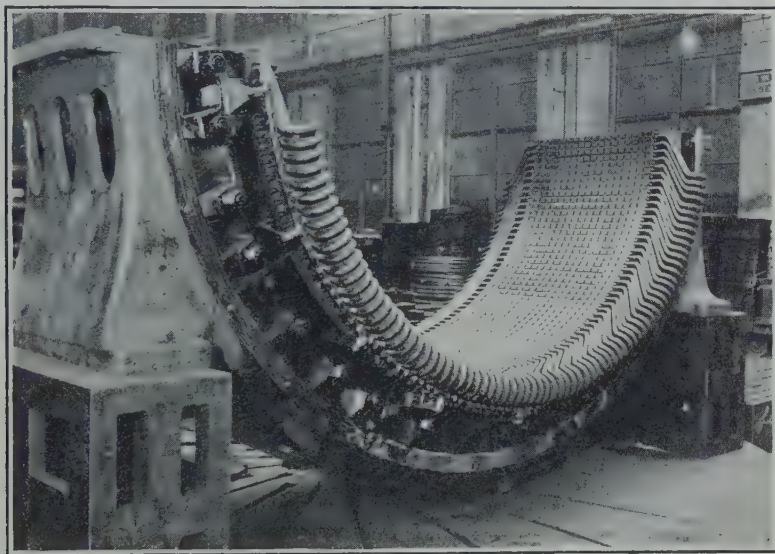


FIG. 187.—Lower half of generator stator—Westinghouse Electric and Manufacturing Company.

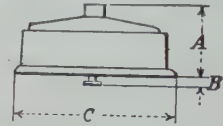
omitted. The thrust bearing must carry the weight of the generator rotor, the water-wheel rotor and also the unbalanced thrust of the water. Its construction is one of the most important mechanical features of the vertical generator (see Chap. IV, page 236). An oil pump, generally geared to the main shaft, is provided to supply oil constantly under pressure to this bearing, and the oil in the reservoir surrounding the bearing is generally cooled by water piping. The guide bearings are also supplied with oil under pressure. The oil from all the bearings is collected at the bottom of the machine, filtered, and returned to the pump.

Brakes are usually provided on a water-wheel-driven alternator to bring it to rest after being shut down, as without brakes slight leakage through the gates might cause the machine to rotate indefinitely. The brakes also constitute an additional safeguard against running away.

Weights and Dimensions.—Weights and dimensions of typical vertical water-wheel-driven generators without direct-connected exciters are given in Table 78. Additional dimensions in somewhat greater detail are given for certain generators in Table 79.

TABLE 78.—WEIGHTS AND DIMENSIONS OF VERTICAL WATER-WHEEL-DRIVEN GENERATORS

Three-phase, 60-cycle, 0.80 power factor, 50-deg. generators with exciters. Efficiencies and excitation are given at $\frac{3}{4}$ load, 0.80 power factor.



Capacity, kilovolt- amperes	Speed, r.p.m.	WR ²	Excita- tion, kilowatts	Shipping weight, pounds	Efficiency, per cent	Dimensions, inches		
						A	B	C
62.5	100	30,600	10	16,500	82.9	98	18	106
	300	33,000	3.5	8,000	87.2	60	12	59
	400	19,000	3.0	7,000	87.9	56	12	59
	900	220	3.0	5,100	88.4			
125	100	61,300	11.0	23,400	84.5	98	18	106
	300	6,700	3.9	11,300	88.2	61	12	68
	400	3,700	3.3	9,900	88.9	62	12	59
	900	445	3.0	7,200	89.4			
625	100	306,000	20.0	52,200	90.9	92	41	156
	300	33,400	10.5	25,300	92.5	88	18	83
	400	18,700	9.0	22,000	92.9	73	12	77
	900	2,250		16,000	93.2			
1,250	100	613,000	29.0	73,900	92.5	100	48	173
	300	66,800	17.5	35,800	93.7	102	25	101
	400	37,100	16.5	31,100	94.0	92	20	83
	900	4,450		22,600	94.1			
1,875	100	919,000	35.0	90,200	93.2	104	52	187
	300	100,000	23.0	43,800	94.3	107	27	101
	400	55,600	21.0	38,100	94.5	103	23	83
	900	6,700		27,600	94.6			
3,000	100	1,470,000	44.0	114,300	94.4	113	59	201
	300	160,000	27.0	55,200	94.8	113	28	116
	400	89,000	26.0	48,100	94.9	110	28	101
	900	10,700		35,000	95.0			
5,000	200	755,000			95.0			
7,500	200	1,270,000			95.4			
12,500	200	2,475,000			96.1			
25,000	200	6,050,000			96.7			

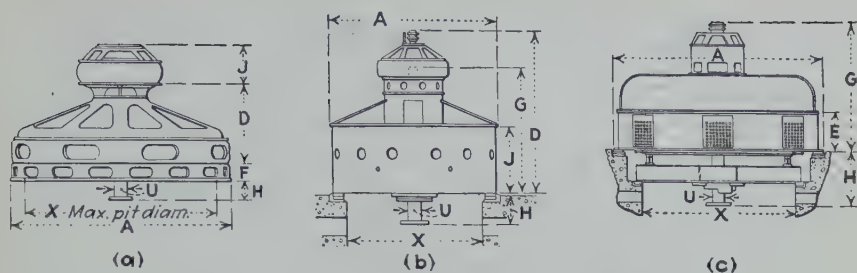


TABLE 79.—DIMENSIONS OF VERTICAL GENERATORS
2300 volts, three phase, 60 cycle

Figure number	Kilovolt-amperes	R.p.m.	Dimensions, inches								
			A	U	G	H	D	E	F	X	J
(a)	62.5	400	59	4½	...	12	33	..	8	51	15
	625	400	77	6	...	12	46	..	8	68	19
(b)	62.5	100	83	5	64	16	85	..	..	74	27
	1875	400	83	8	82	23	87	..	..	82	27
	3000	400	101	9	85	28	110	..	..	86	42
(c)	625	100	156	9	92	41	...	24	..	112	
	1875	100	187	12	105	52	...	31	..	132	
	3000	100	201	14	113	59	...	35	..	144	

At high speeds the rating is limited by the strength of available material, while at low speeds the rating is limited by the physical size that can be accommodated in available machine tools and shipping facilities.

A wide variation is found in the speeds and consequent weights and dimensions of water-wheel-driven alternators as compared with those driven by steam turbines, because the speed of the steam turbine is determined by factors within the control of the designer, while the speed of the water wheel depends largely on the available head.

Rating.—The complete rating of an alternator as given on its rating plate includes output, power factor, voltage, phase, frequency, speed, and current. The maximum output is limited by the allowable maximum temperature which the insulation will safely withstand. Standardized temperatures for various kinds of insulation are given in the Standards of the A.I.E.E. As the heating depends largely upon the current, which varies inversely as the power factor, the rating of an alternator is always given in kilovolt-amperes.

The power-factor rating is determined by the lowest power factor at which the machine will be required to operate. It is generally 80 or 90 per cent. If operated at a lower power factor than rated, the alternator will not carry its rated load without exceeding the rated

temperature rise, but, if operated at higher power factor than rated, it will carry more than the rated load without exceeding the rated temperature rise, because at a low power factor the field current and, hence, the total heating are greater than for the same load at a higher power factor.

The terminal voltage of an alternator is determined by the arrangement and connection of the armature conductors, the speed of rotation, and the field strength. Alternators are ordinarily wound for any standard voltage from 2300 up to 13,800 volts, regardless of kilovolt-ampere capacity. The upper limit is practically set at present by cost of insulation and risk of failure. Lower voltages, such as 6600 or 11,000, are used in more conservative designs.

Performance.—The true efficiency of a generator is the ratio of its useful output to its total input. Its determination involves the accurate measurement of the output and simultaneous input or the accurate measurement of all the losses, which is impossible without the use of highly refined laboratory instruments and means of driving the machine at full load and absorbing its output. Hence, a “conventional efficiency,”¹ which is readily obtained by measuring the principal losses with commercial instruments and which is very close to the true efficiency, is universally used. The efficiency depends upon the size, speed, and general design, and also upon the load and power factor at which the machine operates. Approximate efficiencies of typical vertical alternating-current generators are given in Table 78.

For additional information regarding performance characteristics of alternators, the standard textbooks and handbooks on this subject, a list of which is given at the end of this chapter, should be consulted.

Ventilation.—With most of the modern generators, the disposal of the electrical losses, which appear as heat, presents a serious problem. Ventilation must be depended upon for the most of the cooling, and, with this end in view, the generator is provided with fan blades mounted on the rotor. There are several systems of directing the cooling air through the machine known as the radial, the axial, the circumferential, and other systems, but all essentially take air in at one or both ends, force it through ducts between windings and between sections of the core, and discharge it again at the periphery.

Ducts (which should be short, direct, and with rounded corners) and dampers are often provided so that the air can be taken from out of doors or from the station and can be discharged out of doors or into the station, thereby providing a flexible means of maintaining comfortable station temperature during all seasons of the year. If the ducts are of high resistance, an external fan may be required. Roughly, 100 cu. ft. of air per minute will be required for each kilowatt of loss in the generator,

¹ Standards of A.I.E.E.

and the velocity in the ducts should not exceed 1000 to 1500 ft. per minute.¹

EXCITERS

The Excitation System.—The alternator requires excitation of its field winding from an external direct-current source to maintain the necessary intensity of magnetic flux. The amount of excitation depends upon the speed, the load, and the power factor, being much greater with low speed, lagging power factor, and heavy load. Approximate excitation requirements of vertical water-wheel-driven alternators are given in Table 78.

The principal requirements of an excitation system are reliability, low first cost, economy of operation, simplicity, and convenience. The exciters should be of good design and liberal size, the method of drive should be reliable, and the wiring should be short and simple and carefully installed, and the method of control should be convenient and simple, and with reserve capacity.

The centralized excitation system involving a bus fed by one or more exciters and feeding all the generator fields has the advantage that a minimum number of large exciters may be used and also that a battery may be floated on the bus as an emergency source. The principal objection to the centralized system is the possibility of a ground or other disturbance in one part affecting the entire system. A centralized system should have three identical exciters, one being held as a reserve.

The individual excitation system (one with each generator) has the advantage that each generator with its exciter constitutes an independent unit, and the exciter connections can usually be made shorter and simpler. With the individual system, for a station having a large number of small generators, the exciters may, however, be comparatively small and inefficient.

Motor drive is convenient, inexpensive, and fairly efficient. Induction motors are simple, convenient, and reliable, although synchronous motors may be used if they are specially designed for stable operation during system disturbances. The source of energy to drive the motors must be uniform and reliable.

Water-wheel drive is relatively unreliable, expensive, and inefficient for small individual exciters but is quite feasible for the larger exciters of a centralized system. It is sometimes desirable that at least one exciter be driven by a prime mover for purposes of starting the station. Dual drive, *i.e.*, by means of a motor on one end and a water wheel on the other, is also often used to advantage.

Direct drive from the main unit is very efficient, reliable, convenient, and, except for very slow speed units, inexpensive. Possible objections

¹ RUSHMORE and LOF, "Hydroelectric Power Stations," pp. 342-348.

are that the generator voltage is particularly sensitive to speed fluctuations and that trouble with an exciter may cause the retirement of a large main unit.

Excitation Wiring.—As the output of the generators is essentially supported by the excitation system, it is vitally important that the latter should be thoroughly reliable. The wiring should be short, compact, of ample capacity, well supported, and carefully safeguarded. The insulation should be designed for eight to ten times the normal excitation voltage to take care of transient pressures due to generator short circuits.

Breakers and rheostats should preferably be operated electrically, as they can then be located most advantageously in the station to give short connections. The generator field breaker should be non-automatic but may be arranged to be tripped by generator relay action. The exciter main breaker is usually made automatic on reverse power, in order to disconnect the exciter in case of internal failure when operating in parallel with another exciter. It is not considered advisable to place overload protection on exciters, as it is better to risk injury to the exciter than unnecessary operation of the automatic device.

The excitation circuit should never be suddenly opened. The excessive voltage that would be built up by the sudden releasing of the energy stored in the inductive field winding might be sufficient to puncture the field insulation. To guard against this, the field breaker, which alone should be used to break the circuit, is provided with discharge clips to be connected to the discharge resistor which is always furnished with the generator. As the breaker opens, it connects the resistor across the field terminals and the energy is dissipated in the resistor.

Exciter Construction.—An exciter is a direct-current generator designed especially to supply excitation for synchronous machines. It consists of a stationary field structure within which rotates an armature and commutator on a common shaft. The induced electromotive force is alternating but is rectified by the commutator, so that the output is direct current. The motor-driven or turbine-driven exciter generally has two bearings, but the direct-connected exciter is usually provided without bearings, the armature and commutator being attached to the shaft of the main unit. A direct-connected exciter of this type is shown in Fig. 185.

TRANSFORMERS

Classification.—The essential features of a transformer are (1) a primary winding which receives energy from the supply circuit, (2) a secondary winding which delivers energy to the receiving circuit, and (3) an iron core common to both windings. The windings are so arranged

on the core that alternating current in the primary induces a magnetic flux in the core which in turn induces an alternating current in the secondary. Figure 188 shows the exterior of a typical oil-insulated self-cooled single-phase power transformer.

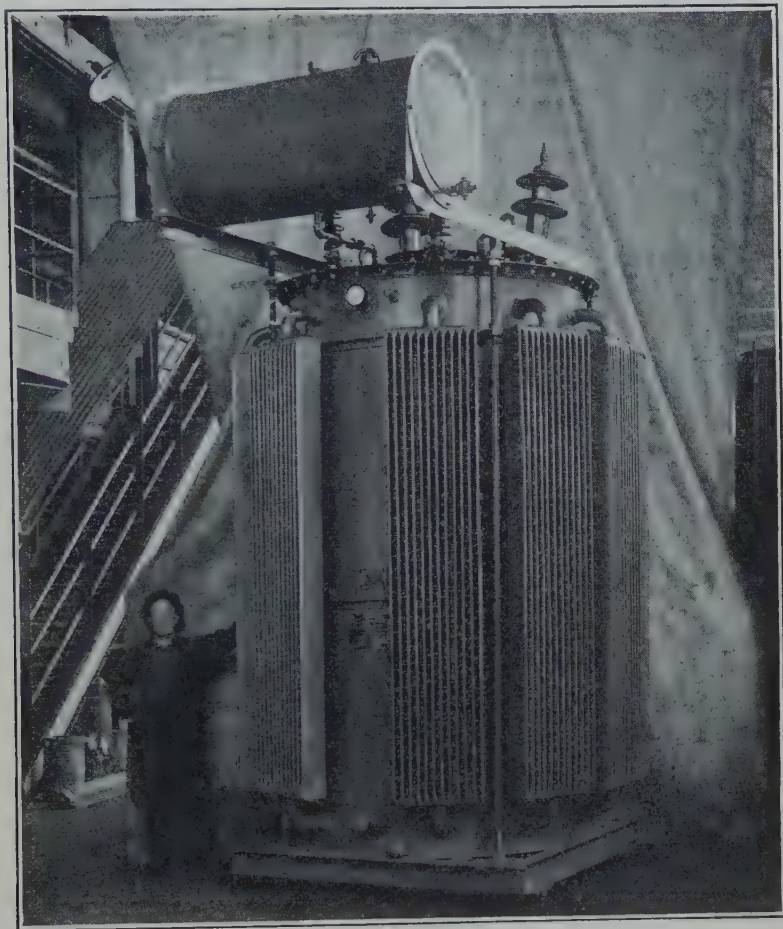


FIG. 188.—Fifteen hundred kilovolt-ampere single-phase transformer—Pittsburgh Transformer Company.

The winding having the greater number of turns is connected to the high-voltage circuit and that with the lesser number of turns to the low-voltage circuit. The ratio, unless otherwise specified, is understood to mean the turn ratio. Without impedance, the voltage ratio would always be equal to the turn ratio, and the current ratio would be the reciprocal of the turn ratio. Due to impedance, however, the primary voltage and current are slightly higher than the values obtained by applying the turn ratio to the secondary voltage and current.

The constant-potential transformer, which at all loads has a constant voltage ratio, except as slightly modified by losses, is the type that is commonly used for transmission and distribution of power.

A single-phase transformer as distinguished from a polyphase transformer has a single primary winding and a single secondary winding. A diagrammatic horizontal section through a single-phase transformer

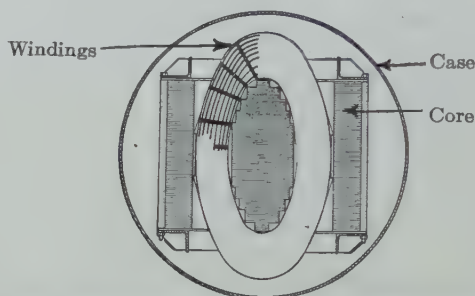


FIG. 189.—Diagrammatic horizontal section, single-phase transformer—Pittsburgh Transformer Company.

is shown in Fig. 189. It can be used alone in a single-phase system or banked with other similar transformers in a polyphase system. The three-phase transformer is the only polyphase type in common use, with three primary and three secondary windings arranged on a common core (see Fig. 190). Advantages of the single-phase transformer are that (1) banks of larger size can be shipped, (2) a defective transformer, can be replaced more easily, (3) a single unit will suffice as a spare for several

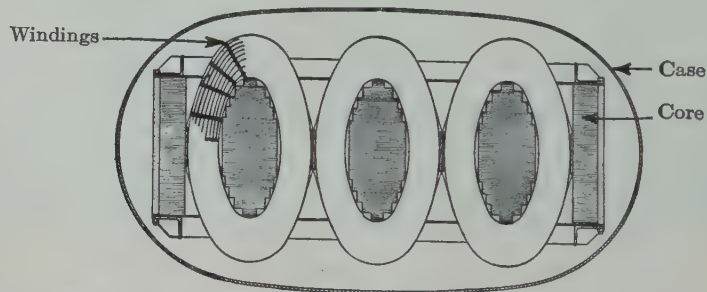


FIG. 190.—Diagrammatic horizontal section, polyphase transformer—Pittsburgh Transformer Company.

banks, and (4) by various groupings many different kinds of transformation may be obtained. Advantages of the three-phase transformer are that (1) the external connections are simpler, (2) less floor space is required, (3) the total weight is less, and (4) the first cost is less.

With the larger sizes and high voltages, it is customary to make all power transformers suitable for either outdoor or indoor service.

Construction.—Transformer cores are built up of thin sheets or laminations of special transformer steel having high magnetic permeability. To prevent eddy-current losses, the laminations are varnished before assembly, or otherwise insulated from each other and from the bolts or clamps which bind them together. The core is built up into

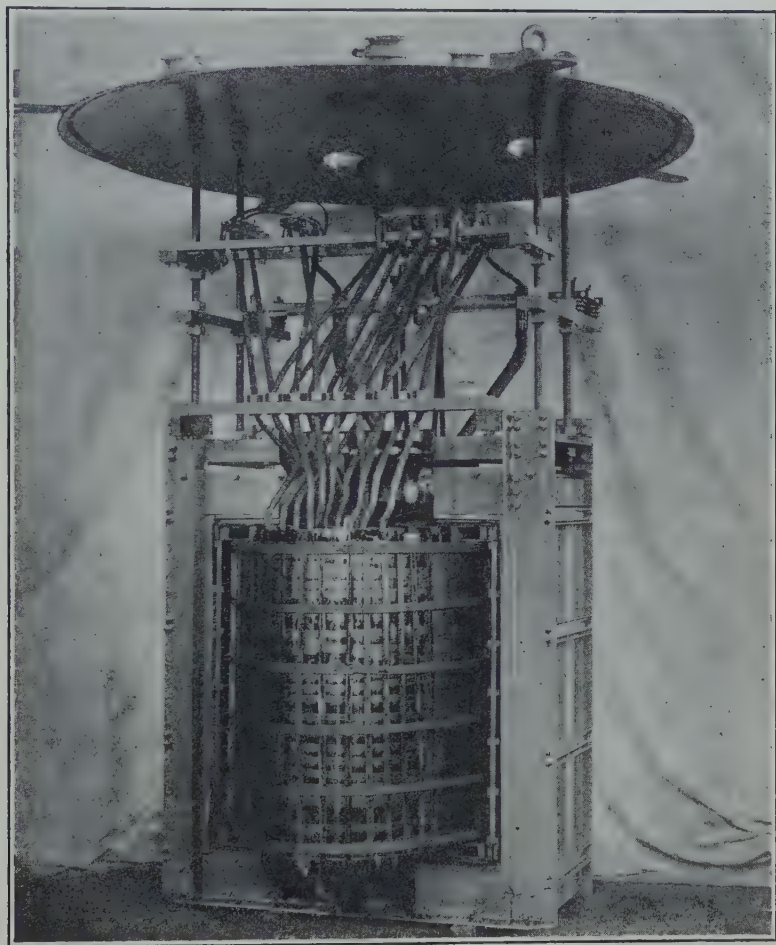


FIG. 191.—Three hundred thirty-three kilovolt-ampere single-phase transformer, case and bushings removed—Pittsburgh Transformer Company.

one of two general types, the core type and the shell type. The latter type is shown in Figs. 189 and 190. Each type has particular advantages as to winding space, insulation, and bracing, and the choice depends upon the capacity, voltage, and frequency.

The windings are of two general types, cylindrical assembled concentrically and discoidal assembled interleaved. The former type is shown

in Figs. 189 and 190. The choice depends upon the capacity, voltage, and frequency. Often the two types are combined in various ways.

Figure 191 shows a single-phase power transformer with core, windings, and cover removed from the case. The connections are shown rising from the windings to the terminal board and bushings. The latter are not shown.

Insulation is one of the most important features of the transformer construction. In the oil-insulated transformer, the core and coils are immersed in oil which serves as both an insulating and a cooling medium.

Bushings for bringing the leads out through the case are of three general types, the solid type, the condenser type, and the oil-filled type.

The case or tank is usually built of boiler plate riveted and welded oil-tight, fluted for the self-cooled transformer and sometimes provided with external radiators as shown in Fig. 188. For water cooling the case is smooth. Cooling coils should be arranged to drain themselves when the water supply is shut off, to prevent freezing, and should be of such shape that the core and windings can be lifted out of the tank without disturbing the coils. The case should be provided with wheels or castors, lifting and pulling lugs, thermometer, oil-drain valve, sampling valve, filtering connections, and oil gage.

Conservation of the oil to exclude moisture and oxygen is of great importance, as moisture very greatly reduces the insulating properties of the oil, and the presence of oxygen, together with high operating temperatures, causes the oil to sludge. Various methods have been developed for eliminating moisture and oxygen from the air breathed into the case as a result of expansion and contraction, including the chloride of lime breather, the conservator or expansion tank transformer (shown in Fig. 188), and the "inertaire" transformer.

Rating.—The rated output of a transformer is the product of the volts and amperes at its secondary terminals when delivering its maximum continuous load and is expressed in kilovolt-amperes. The maximum continuous output is limited by the heating. The standards of the A.I.E.E. specify the limiting temperatures for various classes of insulation and the methods of measuring the temperatures.

The rated primary voltage, as defined in the standards of the A.I.E.E., is the rated secondary voltage multiplied by the turn ratio. Hence, when the transformer is carrying load at rated secondary voltage, the primary voltage is higher than rated by an amount equal to the regulation. The test voltage specified in the standards of the A.I.E.E. is twice the normal voltage of the circuit to which the transformer is connected, plus 1000 volts at not less than rated frequency and for a period of 60 sec.

Performance.—The efficiency of a transformer is the ratio of the kilowatt output to the kilowatt input. The losses are classified as no-load losses, or those which continue as long as the transformer is energized,

regardless of whether it is delivering any load, and the load losses which are dependent on the load delivered.

It is necessary, when giving the efficiency of a transformer, to state the load and power factor at which it applies. It is usual for the manufacturer to submit with his quotation the efficiencies at several loads and at unity power factor. The full-load, 100 per cent power factor of the modern 60-cycle power transformer varies from 96 to 99 per cent depending on size and voltage. The 25-cycle transformer is slightly less efficient.

By properly proportioning the iron and copper losses, the manufacturer can usually obtain the highest efficiency at 50, 75, or 100 per cent load as desired to give the best all-day efficiency.

The regulation of a transformer is the difference between the no-load secondary voltage and the rated-load secondary voltage expressed in per cent of rated secondary voltage, the primary voltage being constant at such value as to give rated secondary voltage at rated load. The regulation varies with the power factor of the load, and, therefore, the power factor at which the regulation is given must always be stated.

Parallel operation of two or more transformers, or two or more banks of transformers, with distribution of the total load between them in proportion to their respective ratings, requires that they have equal voltages and turn ratios and equal per cent impedances. If the ratios of the transformers are not equal, one may be corrected by use of an auto-transformer connected in series. If the impedances are not equal, one may be corrected in emergencies by use of an external impedance connected in series. The manufacturer, if provided with data on the existing transformers, can nearly always design the new transformers to operate satisfactorily in parallel with the old ones.

Cooling.—The losses of the transformer appear as heat which must be carried away as fast as formed so as to prevent excessive temperature rise. The method of cooling classifies the transformer in one of four general types which are, in order of importance: self-cooled, water cooled, forced-oil cooled, and air blast.

The self-cooled transformer has the core and windings immersed in oil, and the case is either fluted or provided with radiators so that the oil is cooled by natural radiation. The oil circulates by convection upward through the hot windings and downward through the flutes or radiators. This type can be built in practically any commercial capacity and voltage. With the very large capacities, however, the radiators become larger than the transformer cases themselves and, therefore, bulky and expensive. The great advantage of this type is its simplicity.

The water-cooled transformer is also oil insulated, but the tank is plain. Around the inside of the tank at the top where the oil is hottest, a cooling coil is placed, through which cold water is circulated. It is

desirable to use a low water pressure to prevent possibility of water leaking into the oil, and for this reason it is customary to discharge the water from the transformer at or below atmospheric pressure.

The forced-oil-cooled transformer is similar to the water cooled, except that it has no water coils. The oil is circulated by pumps through external radiators which are cooled by air, or through surface coolers which are cooled by water. Oil circulation should be upward through the transformer and downward through the cooler; water circulation upward through the cooler. It is important that the oil pressure be maintained higher than the water pressure to prevent water leaking into the oil, as a mere trace of water in the oil might prove fatal to the transformer. Purity of the water is not so important as with the water-cooled transformer, for the cooler can be made in sections and part at a time can be cleaned and inspected.

Oil.—Oil for transformers is obtained from crude petroleum by fractional distillation. Desirable qualities are (1) high resistivity and dielectric strength, (2) low viscosity, (3) high flash and burning points, (4) high thermal conductivity and specific heat, (5) chemical neutrality toward metals and insulating materials, and (6) chemical stability at high temperatures.

Moisture even in very minute quantities very greatly reduces the dielectric strength of oil. There are two common methods of drying and purifying oil: (1) by passing it through a centrifuge, and (2) by forcing it through dry blotting paper.

SWITCHING EQUIPMENT

Knife Switches and Fuses.—Knife switches are used for controlling excitation, auxiliary power, lighting, and signal circuits. They are available in ratings of 0 to 600 volts and 0 to 3000 amp. and in a variety of forms such as one, two, three, and four pole; single and double throw; front and back connected; top or bottom fused or unfused; punched and milled clip. When mounted on switchboards or panel boards, the blades should always be mounted vertically with hinges at the bottom, except that with double-throw switches the blades should be mounted horizontally. The blades should be "dead" when open. It is often advisable to provide 90-deg. stops to limit the opening of the switches where the panel is crowded. For 600-volt circuits, switches are often provided with quick breaks and with barriers.

Fuses are of three general types, enclosed N.E.C.,¹ plug, and open link. Those for use with knife switches are preferably of the enclosed N.E.C. type. Plug fuses are sometimes used in lighting cabinets. Open-link fuses are used principally for heavy circuits and in locations where the blowing of a fuse cannot damage surrounding equipment.

¹ National Electrical Code, National Board of Fire Underwriters, New York.

A knife switch enclosed in a steel box arranged to be operated by a handle from the outside, commonly known as a "safety switch," is a form of switch extensively used throughout the station at motors, on cranes, and other places apart from the switchboards. It may be fused or unfused as desired. For safety the box is generally constructed so that the switch must be opened before the fuses can be reached and so that the switch can be locked in the open position.

The field switch which is used to control the field current of a generator or other machine is designed with a discharge clip which closes while the switch is being opened, so as to discharge the inductive energy from the field through a field discharge resistance. On account of the flash that usually results from the opening of the field switch, the switch is generally placed on the back of the board and operated by a lever. It is sometimes provided with arcing tips and barriers. In large stations the field switches consist of electrically operated carbon circuit-breakers mounted near the generator. The field transfer switch is a double-throw switch with extra long clips arranged so that the field can be thrown from one bus to another without being opened.

Disconnecting Switches.—A disconnecting switch is a form of knife switch used for isolating oil circuit-breakers, generators, transformers, arresters, and other equipment, as a selector switch in conjunction with double buses and single circuit-breakers, and as a tie switch between circuits. It is not designed to be opened under load. A disconnecting switch usually consists of a single-pole knife switch arranged with an eye to receive a switch hook by means of which it is opened and closed. It is mounted on porcelain insulators and suitable marble or metal base. A latch to prevent the switch from flying open under short-circuit current is necessary except on very small power systems.

For outdoor use, disconnecting switches are sometimes provided with horns to direct the arc upward and are then capable of breaking the charging current on short lines or small transformer banks. They are then called horn-gap switches or pole-top switches.

Carbon Circuit-breakers.—Carbon circuit-breakers are used most extensively on direct-current circuits such as the excitation and battery circuits in the power station and the direct-current motor generator, rotary converter, and feeder circuits in the substation. They are available in voltages up to 750, in ampere capacities up to 10,000 in one-, two-, three-, and four-pole types, and for operation manually and electrically. A typical carbon circuit-breaker is shown in Fig. 192.

Oil Circuit-breakers.—Oil circuit-breakers are used principally for controlling alternating-current circuits. They can be obtained in ampere capacities from 50 to 4000; for voltages from 220 to 220,000; one-, two-, three-, or four-pole; manually or electrically operated; and for indoor or for outdoor service. The selection of the proper breaker

for each circuit in a power system involves consideration of several very important factors such as normal voltage, normal current, abnormal current, interrupting duty, method of operation, arrangement of terminals, space available, accessibility for repairs, altitude of installation above sea level, and temperature.

A breaker is rated in accordance with the standards of the A.I.E.E. on the basis of (1) the normal r.m.s. voltage of the circuit on which it is intended to operate, (2) the normal r.m.s. current it is designed to carry continuously with a temperature rise of the oil and contacts by

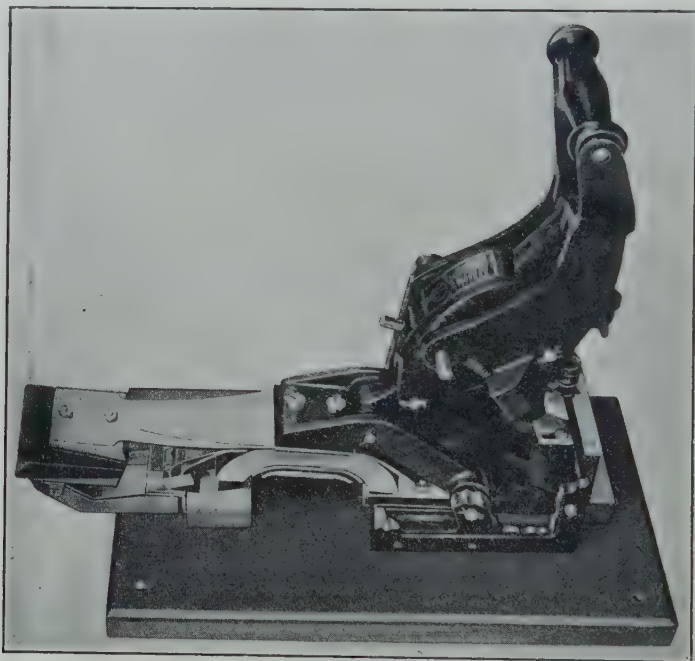


FIG. 192.—Carbon circuit-breaker. Single pole, hand operated—The Cutter Electric Company.

thermometer of not over $30^{\circ}\text{C}.$ above an ambient temperature of $40^{\circ}\text{C}.$, (3) the normal frequency of the current, and (4) the maximum r.m.s. current at normal voltage which it can interrupt under prescribed conditions at stated intervals a specified number of times. A further rating which is very important for breakers to be operated non-automatically or with a time delay is (5) the short-time rating, based on the maximum r.m.s. current the breaker will safely carry for a short time as 1, 2, 3, or 5 sec. without excessive heating or mechanical injury.

The value of the short-circuit current which a breaker will be called upon to interrupt in a particular location in a system depends upon the connected synchronous capacity on the system, the characteristics of the machines, transformers, circuits, and other equipment, and the time

allowed for the breaker to open after the short circuit occurs, and involves a very detailed analysis of the entire system to which the breaker is to be connected with particular reference to future growth.

Breakers may be mounted on the switchboard panels or panel pipes for voltages not higher than 2500 or for ampere ratings not higher than

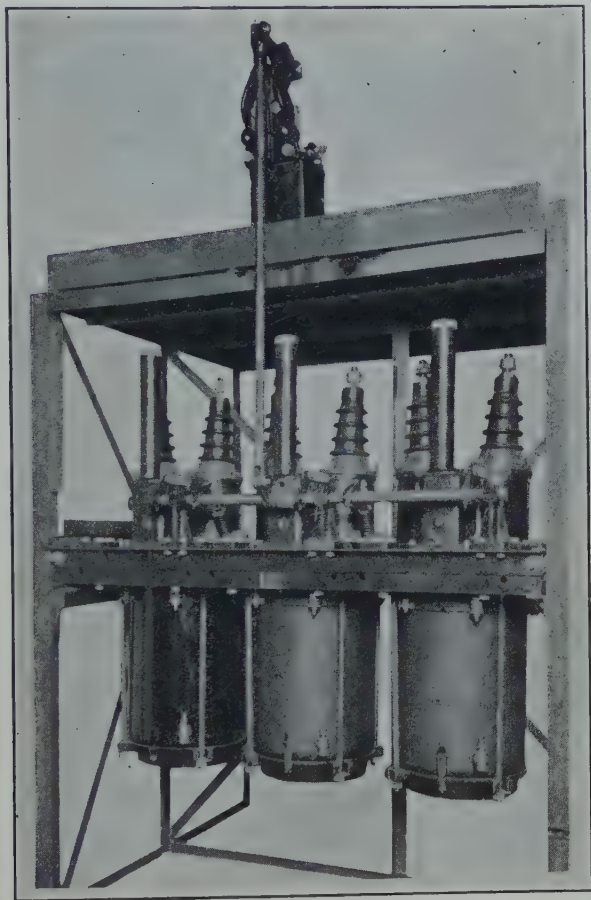


FIG. 193.—Solenoid-operated oil circuit-breaker, 25,000 volts, 600 amp., three pole—General Electric Company.

800. Larger breakers are mounted away from the switchboard on pipe or steel framework or in masonry cells, the latter method being used for the main circuits of all but the smallest of power stations and substations. With the so-called isolated phase arrangement, the three poles of the breaker are placed in separate rooms or on separate floors for more complete isolation. Usually for voltages above 25,000 and occasionally for lower voltages, the breakers are located outdoors, where more space is available and greater clearances can be obtained.

The oil circuit-breaker, as the name implies, is arranged to break the circuit under oil. A large-capacity solenoid-operated breaker mounted in a concrete cell is shown in Fig. 193. A single-pole element of a similar breaker, with the oil tank removed, is shown in Fig. 194. In the smaller breakers the three poles are arranged in a single oil tank, while in medium and large breakers each pole has a separate tank. Auxiliary contacts

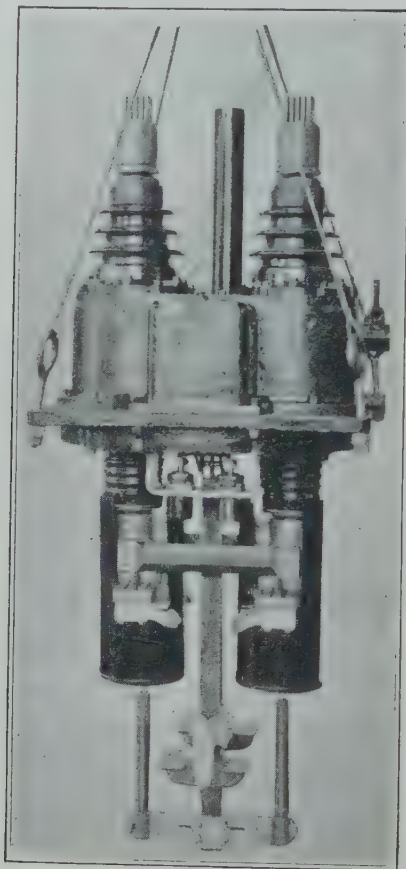


FIG. 194.—Single-pole element of oil circuit-breaker with oil tank removed—General Electric Company.

are provided to open after and close before the main contacts so as to relieve the main contacts of the burning action of the arc. When heavy currents are interrupted by the breaker, the pressure from the heat of the arc is very great and consequently the tank must be made very strong. In some types of breakers, this pressure is utilized to force the contacts open more rapidly. Provision is often made on heavy-duty breakers to vent the tanks to outdoors to allow safe escape for any gases that may be formed by the arc.

Accessories for oil circuit-breakers include hand-closing levers for electrically operated breakers, tank lifting and removing devices, control relay for reducing the duty on the control switch, bushing-type current transformers for relays and ammeters, auxiliary switches for controlling pilot lamps, alarms and interlocks, oil gages, oil-drain and sampling valves, and filter-press connections.

Switchboards.—Switchboards, which contain the control, metering, relay, and sometimes the switching equipment of the station, may be designed in several forms such as upright, bench or desk, dead front, truck type, straight, and curved. The selection depends upon the kinds and capacities of apparatus to be controlled, style of switchboard devices to be used, and space available in the station. The main circuits are generally controlled from the main control board, with remote, electrically operated breakers. The auxiliary circuits may be controlled in a similar manner, but it is often preferable to place the breakers for the auxiliary circuits directly on the auxiliary switchboards.

The main control and instrument board of the power station or substation should be located in a quiet, well-lighted and well-ventilated room for the comfort of the operators who are always in attendance. The board is generally of the upright or bench type, or a combination of the two types, and is sometimes arranged on an arc of a circle to assist the operator in seeing the entire board from one position. The excitation circuit-breaker boards, station light and power boards, battery board, and others which do not require constant attendance may be located about the station to secure best economy and convenience. These boards are often of the truck type to provide maximum safety and convenience. If of the open type, ample space should be provided for personal safety of employes, both for normal operation and for maintenance and repairs. For all voltages over 150, provision should be made for retiring any portion for working on it without interfering with station operation.

The framework for a switchboard consists of a sill, uprights, and braces. A suitable sill consists of a 6-in. steel channel bolted and grouted to the floor. The uprights consist of $1\frac{1}{4}$ -in. standard black-iron pipes or angle irons, braced to the wall or adjacent structure by similar pipes or angles. The space behind the board is usually enclosed by steel grilles with at least two access doors.

Panels are made of slate, marble, ebony-asbestos compound, or steel, slate being most commonly used. Slate does not have particularly good insulating qualities, however, and should never be used for over 1200 volts. Even for 100 volts it must be carefully selected to avoid metallic veins. Marble may be used for voltages up to 3500 and is consequently used extensively for 2300-volt disconnecting switch bases. Marble gives a good finish but is easily spoiled by oil stains and is very

difficult to match. Ebony asbestos is considerably stronger than either slate or marble and can be safely used with voltages up to 4500. Its texture and color are uniform and it can be finished almost as attractively as can slate. Steel has the advantage of lightness and strength. All current-carrying parts must be insulated from the steel, but this presents very little difficulty on a control or a truck board.

Equipment for switchboard panels should always be located with respect to its use. Voltmeters, synchrosopes, power-factor indicators, ammeters, and wattmeters must be within sight of the operator. Integrating and curve-drawing meters and relays are usually located near the bottom of the board or, preferably on the rear of the board. Control switches and all devices that require operating must be placed within reach. It is for the purpose of obtaining more space for such equipment that the bench board or control desk is extensively used. This may be extended in a vertical section 10 or 12 in. high at the rear edge to provide still more space within reach of the operator.

Instrument Transformers.—Instrument transformers are used wherever instruments, meters, relays, and such equipment are to be connected to circuits of heavy current or high voltage. Their purpose is twofold: (1) to insulate the instruments from the main circuit as a safeguard to the employees, and (2) to reduce the current or voltage to a value suitable for the instruments.

The current transformer is used to reduce the current of the main circuit for ammeters, wattmeters, power-factor indicators, relays, and other instruments having current coils. It is so designed that when the instrument load, or secondary volt-ampere burden as it is called, is within certain limits for which the transformer is designed, the phase angle and current in the secondary are closely proportional to those in the primary. The primary usually consists of one or more turns of large conductor which becomes a part of the main circuit, and the secondary consists of a larger number of turns of smaller wire which is connected to the instrument circuit. Except as affected by the losses, which are small, the currents are inversely proportional to the numbers of turns. The secondary current is usually 5 amp. The laminated core and secondary winding are carefully insulated from the primary. In the larger current capacities, there is no primary winding, the core and secondary being arranged to slip over the main conductor which thus becomes in effect a one-turn primary. This type is often used in the bushings of high-voltage oil circuit-breakers and is known as a bushing-type current transformer. Single-turn current transformers do not have a high degree of accuracy when used on circuits of small ampere capacity and for this reason are not recommended for use with wattmeters and watt-hour meters, although they are sufficiently accurate for ammeters and relays. For accurate measurement on a high-voltage,

low-ampere circuit, it is necessary to use an oil-insulated current transformer with several primary turns.

The potential transformer is used to reduce the voltage of the main circuit for voltmeters, wattmeters, power-factor indicators, and other instruments having potential coils. Like the current transformer, it is designed for a limited range of volt-ampere burden. The potential transformer is similar in construction to a power transformer but of small size, usually 200 watt. Except as affected by the losses, which are small, the voltages are proportional to the number of turns. The secondary voltage is generally 110 volts. Below 10,000 volts the transformer is usually of the dry type, while for higher voltages, it is usually oil insulated. The potential transformer is usually protected by a primary fuse and sometimes also by a primary resistor.

Reactors.—Reactors are used in power systems primarily to reduce short-circuit currents to values that can be safely carried by the apparatus and interrupted by the circuit-breakers. They are also used to some extent to maintain bus voltage during short circuit, to parallel dissimilar transformers, to allow changing transformer taps under load and as choke coils.

The correct application of current-limiting reactors requires a thorough analysis of the short-circuit characteristics of the system. The reactors may be placed in the generator circuits, in the bus between generators or in the feeders. As they cause some power loss and voltage drop, it is preferable to locate them between machines where they do not normally carry heavy current, rather than in the main circuits.

The reactor consists essentially of an inductive coil without an iron core. It must be insulated from ground sufficiently to carry line voltage and must have sufficient carrying capacity to carry line current. As the mechanical forces are very severe in case of short circuit, the coils of the reactor must be very substantially supported, and the reactors themselves, unless ample space is left between them, must be firmly braced against each other. Figure 195 shows a single-phase reactor, three of which are required for a three-phase circuit. The particular type of reactor shown is a coil of insulated copper cable partially embedded in a heavy concrete frame. The conductor may be supported in specially designed porcelain tiles, or entirely embedded in a hollow cylindrical concrete shell. The principal requirements of the design are (1) insulation, (2) mechanical strength, (3) resistance to heat, (4) elimination of iron parts and of through bolts which might cause short circuits, and (5) ease of inspection.

The rating of the reactor is usually stated in percentage of normal voltage represented by the voltage drop in the reactor with rated current flowing. Thus for a three-phase 13,200-volt 100-amp. circuit a 10 per cent bank of reactors would cause a voltage drop of 1320 volts in the

circuit when carrying 100 amp. The frequency, current, and voltage must also be stated when fully specifying the reactor.

Voltage Regulators.—Voltage regulation is one of the most exacting problems of the designing or operating engineer. While generators, transformers, and motors will operate fairly satisfactorily over a voltage range from 10 per cent below to 10 per cent above normal, the requirements of heating devices and lamps are much more critical, as shown by

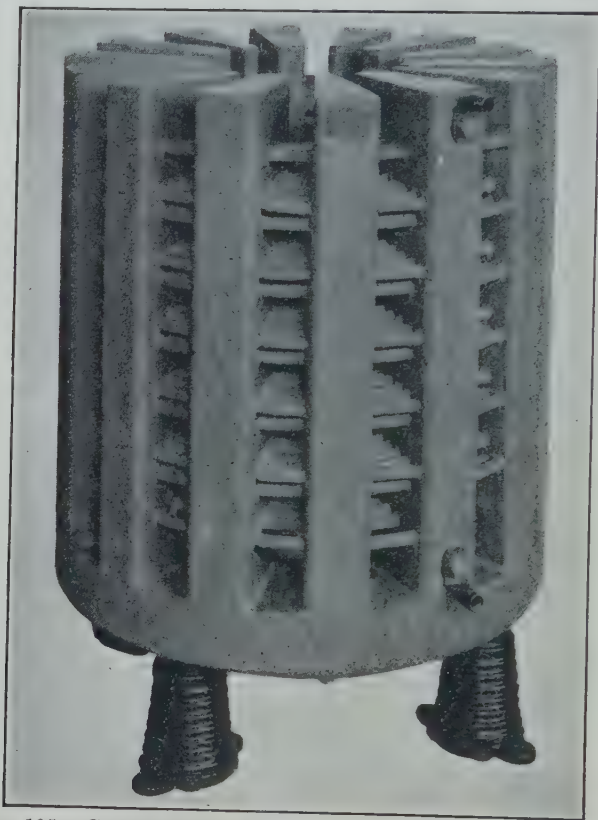


FIG. 195.—Current-limiting reactor—General Electric Company.

the effect of voltage variation on a Mazda lamp in Fig. 196. It is, therefore, very important to regulate the voltage carefully at the customer's terminals.

The problem of maintaining constant voltage would be comparatively simple if the load were constant. The load on the system is constantly changing, however, and each change of load causes changes in voltage due to the voltage drops through generators, transformers, and circuits. To maintain constant voltage at the customer, therefore, it is necessary constantly to regulate the voltage by means of automatic voltage regulators.

Two general types of voltage regulators are commonly used: (1) the generator voltage regulator which regulates the voltage of the generator and (2) the feeder voltage regulator which regulates the voltage of an individual feeder.

The generator voltage regulator most commonly used is known as the vibrating regulator and operates on the exciter field rheostat. Figures

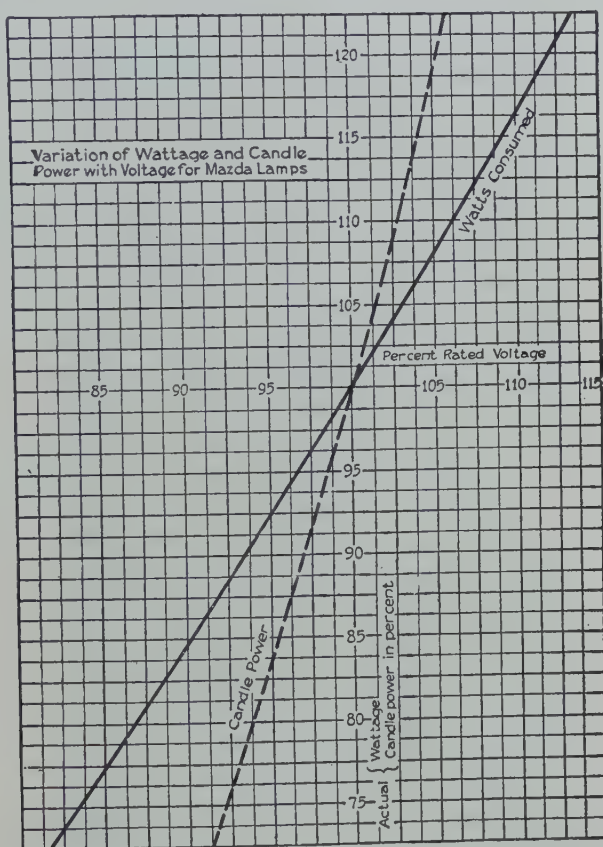


FIG. 196.—Variation of wattage and candle power with voltage for Mazda lamps—General Electric Company.

197 and 198 illustrate a regulator of this type. A shunt connection across the exciter field rheostat is rapidly opened and closed by a series of relay contacts, thus in effect cutting the rheostat into and out of service. The average time the shunt is open as compared with the average time it is closed determines the average exciter voltage, which in turn determines the generator field current and the generator voltage. The contacts which open and close the shunt are themselves controlled by a potential transformer on the generator bus. Thus, a slight drop in bus voltage increases the time the shunt is closed as compared with the

time it is open, the exciter voltage builds up, increasing the generator field current, and normal voltage is restored. A slight rise in bus voltage is corrected by a similar but reverse operation.

When several exciters are operated in parallel on a common exciter bus, one regulator is generally used for the group of exciters. This case

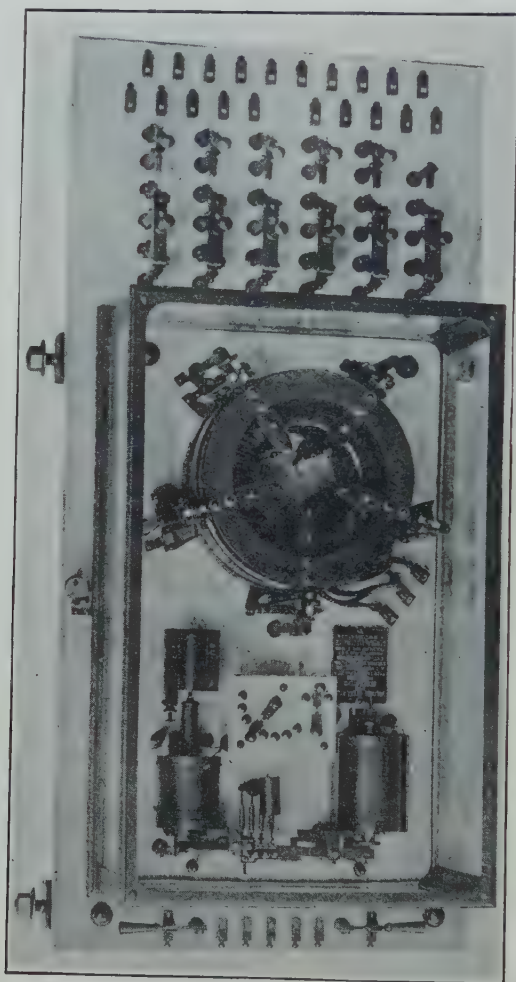


FIG. 197.—Generator voltage regulator—General Electric Company.

is illustrated in Fig. 198. With an individual exciter for each generator a separate regulator is generally used on each exciter, although it is possible to use a single regulator provided the exciters are not too large and provided the generators are paralleled on a common generator bus from which the potential control circuit can be taken.

When alternating-current generators are operated in parallel, changing the field current of one generator does not alter that generator's share

of the load nor does it appreciably alter its voltage. It does, however, alter its share of wattless current, or, in other words, it changes its power factor. In order properly to apportion the station power factor among the various generators and thereby avoid excessive circulation of wattless current between generators, a compensating winding is placed on the control magnet and energized by a current transformer in the generator main circuit. The regulator is, therefore, controlled not only by the bus voltage but also by the generator current and maintains uniform power factor among the various generators as well as normal bus voltage. The

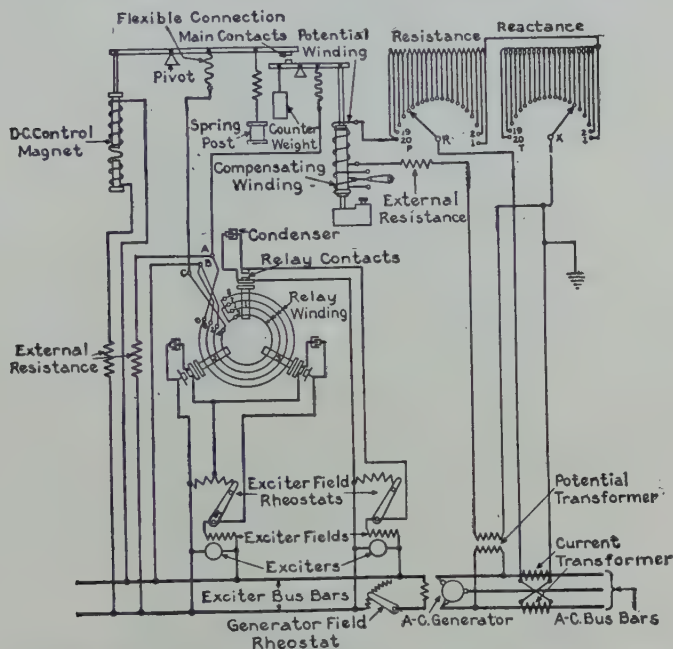


FIG. 198.—Elementary connection diagram of generator voltage regulator—General Electric Company.

compensating winding is shown in Fig. 198, but the current transformer and connections are not shown.

In case it is desired to maintain uniform voltage at a point on the system some distance from the generating station, this could be accomplished by placing the potential transformer at the point and running the potential leads back to the regulator. The leads, however, would be expensive and subject to injury. A more common scheme is to provide the regulator with a line-drop compensator. One form of this device is shown in Fig. 198. It consists of a resistance and a reactance, both adjustable and connected in series between the potential transformer and the regulating magnet. A secondary current from current transformers in the feeder is made to pass through the resistance and reactance, the

secondary current being proportional to and in phase with the feeder current. By proper adjustment of the resistance and reactance, a voltage drop is set up in the potential circuit proportional to the voltage drop in the feeder.

A regulating or voltage-adjusting rheostat is often provided in the circuit between the potential transformer and the regulating magnet, by which the voltage maintained by the regulator may be adjusted within reasonable limits. This is very convenient in case the bus voltage is

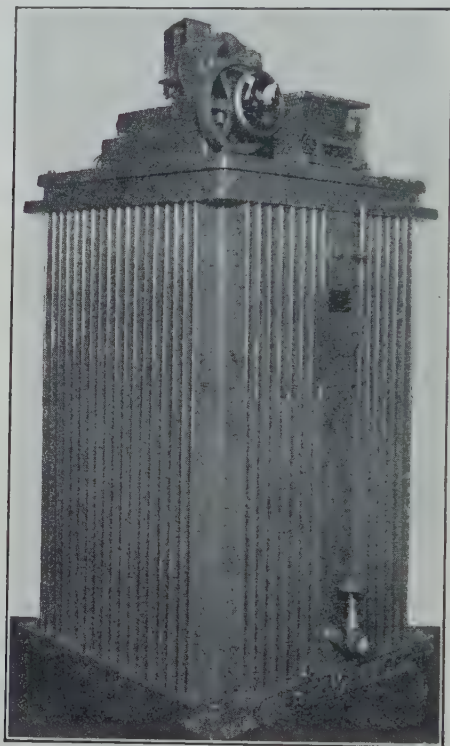


FIG. 199.—Motor-operated three-phase induction regulator—Westinghouse Electric and Manufacturing Company.

desired to be changed slightly during the day to accommodate load conditions. An auxiliary or equalizing rheostat is required in series with each exciter rheostat when exciters are paralleled unless the exciters are exact duplicates of each other. These rheostats are adjusted to cause the exciters to divide the load properly and after this initial adjustment do not require further attention.

With the automatic vibrating-type regulator operating on the exciter fields as described in the preceding paragraphs, the exciter-bus voltage often varies from 40 to 100 per cent of normal. In case it should be desired to operate any of the station auxiliaries from the exciter bus,

such a voltage range would be prohibitive. To meet such conditions, other forms of regulators have been used. One consists of the usual vibrating-type regulator operating on the field of a booster exciter which is connected between the constant-voltage exciter bus and a variable-voltage generator field bus. Another type consists of a dial or face-plate rheostat directly in the generator field circuit and operated automatically by motor or solenoid. This, of course, is wasteful of energy as the generator field current and consequent rheostatic losses are comparatively large.

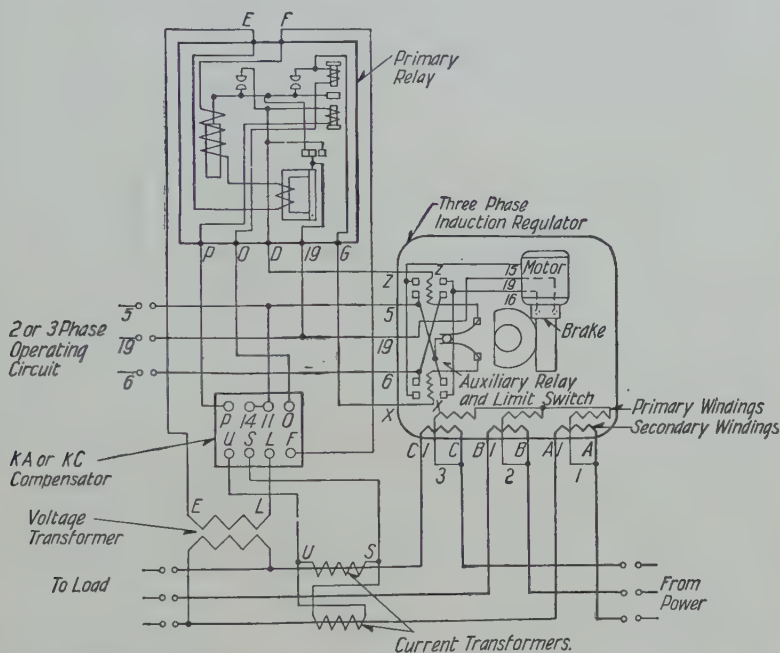


FIG. 200.—Diagram of connections for an automatic three-phase regulator—Westinghouse Electric and Manufacturing Company.

The feeder voltage regulator, the usual form of which is known as the induction voltage regulator, is used to control the voltage on a particular feeder or circuit. In many stations each important feeder is provided with a voltage regulator, as it is impossible to regulate the bus voltage to suit the varied requirements of the separate feeders. Regulators are sometimes used also on tie lines between stations in order to effect a desired interchange of energy without interfering with the normal voltage conditions at each station.

A typical three-phase regulator is shown in Fig. 199 and wiring diagram in Fig. 200. The regulator consists essentially of a variable-ratio transformer, the primary of which is connected across the feeder circuit and the secondary in series with the feeder circuit. The two windings are assembled on separate concentric laminated iron cores, the secondary

being stationary and the primary being arranged to be partially rotated within the secondary. By rotating the primary through 180 electrical degrees, the action of the regulator is gradually changed from one of boosting or raising the feeder voltage to one of bucking or lowering the feeder voltage. The movement of the primary may be accomplished by manual operation or by a manually controlled motor. The most common method, however, is by means of an automatically controlled motor. In this case, which is the one illustrated in Figs. 199 and 200, a contact-making voltmeter is the controlling device. If desired, a line-drop compensator with current and potential transformer may be used to maintain constant voltage at the far end of the feeder in much the same way as described in previous paragraphs in connection with the generator voltage regulator.

The feeder voltage regulator is rated in amperes, volts, and frequency corresponding to the feeder it is to regulate. The single-phase regulator is rated in kilovolt-amperes corresponding to the product of the amperes, the volts, and the percentage rise or lowering. The three-phase regulator is rated in kilovolt-amperes corresponding to the product of the amperes, the volts, the percentage rise or lowering and $\sqrt{3}$.

Feeder voltage regulators may be obtained in cases suitable for either indoor or outdoor installation. They are usually oil filled and cooled in much the same way as transformers are cooled, that is, by radiation, by water coils, or by oil circulation.

Lightning Arresters, Gaps, and Choke Coils.—A lightning arrester is a device for protecting apparatus against lightning and other abnormal potential rises of short duration. It is rated by the voltage of the circuit on which it is to be used.

Arresters are of two general types: (1) the "gap and resistance" type wherein a series gap determines the voltage at which it will discharge and the discharge is proportional to the total applied voltage, and (2) the "valve" type wherein the characteristics of the cells largely determine the voltage at which it will discharge and the discharge is proportional to the excess voltage above a certain critical value.

The gap and resistance type is generally used only on the lower voltages. The simplest and least expensive form consists of a single gap in series with a resistance between line and ground. After the gap breaks down, the resistance limits the current flow, thus helping to extinguish the arc at the gap. A high resistance allows a closer gap setting, and a low resistance allows a more effective discharge. The choice must be a compromise, which results in a sacrifice in efficiency. Another type has a gap, resistance, and magnetic blowout. A lower resistance and closer gap setting can be used, and this type is quite satisfactory, especially for direct-current service. Another type consists of a gap and a fuse in series. It is effective but non-restoring. Another type consists

of a gap, resistance, and self-restoring circuit-breaker in series. It is fairly satisfactory for direct-current service but has the disadvantage that its operation depends upon moving parts and that, while the breaker is restoring itself, the arrester is unprepared for duty. The Bennett arrester consists of a single gap in series with a water-column resistance. The current enters the water column at the top through a carbon electrode and the heat vaporizes a portion of the water, driving part of the water out through an opening in the bottom into another chamber, thereby increasing the resistance and interrupting the arc. Another single-gap arrester, known as the multipath or carborundum arrester, consists of a single gap in series with a block of carborundum. The resistivity and structure of the carborundum cause the discharge to spread over the entire block avoiding concentrated heating and consequent arcing. The condenser-type arrester consists of an electrostatic condenser with or without a series gap. The multigap arrester consists of a series of small gaps in combination with shunting resistances. The gaps occur between a series of relatively large cylinders of non-arcing metal, usually a brass rich in zinc. The low-equivalent arrester consists of multigaps in series with a resistance. The compression arrester consists of multigaps in series with a resistance, the gaps being enclosed in air-tight chambers. The graded-shunt-resistance arrester consists of multigaps bridged by resistors of various values connected between the line end and various points along the series of cylinders.

The valve-type arrester may be adapted to practically any voltage. The aluminum cell or electrolytic is probably the most extensively used and best known of this type, although it has recently been superseded for alternating-current service by other equally efficient types. It takes practically no current at normal voltage, discharges vigorously at relatively slight excess voltage, and immediately restores itself. Essentially it consists of a stack of aluminum trays nested, insulated from each other, partially filled with electrolyte, and finally immersed in a steel tank of oil, thus forming a series of electrolytic cells between line and ground. With the application of rated potential which is about 250 to 300 volts per cell, a film of aluminum hydroxide is formed on each aluminum surface. The film effectively prevents current flow at the voltage at which it was formed, but at temporarily higher voltage it allows a discharge in proportion to the excess voltage. As the film gradually dissolves when gaps are used so that line potential is not constantly applied, arresters with gaps must be charged daily by closing the gaps for a few seconds. This inconvenience and the fire hazard from the oil constitute the principal objections to this type of arrester.

The oxide-film arrester is another form of valve arrester adaptable to all voltages and is offered by one manufacturer in preference to the aluminum arrester. An arrester of this type is shown in Fig. 201. It

consists essentially of a stack of cells in series with a gap. Each cell consists of two brass disks separated by an annular ring of porcelain, the interior space being filled with lead peroxide powder which has a low resistance. A film of varnish on the side of each brass disk in contact with the lead peroxide provides sufficient insulation to prevent appreciable current flow at rated voltage which is about 300 volts per cell. At abnormal voltage the series gap is broken down and the varnish films are punctured. The current flow is proportional to the excess voltage. The action of the current on the lead peroxide is to form red lead and litharge which immediately seal up the punctures restoring the insulating value of the original varnish film, and the arrester is again ready for duty. The pellet-type arrester is a modification of the oxide-film type particularly adaptable to distribution circuits. The lead peroxide is formed into

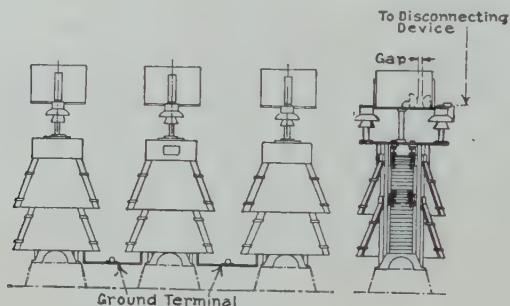


FIG. 201.—Oxide-film lightning arrester.

small pellets which are coated with an insulating powder and placed in a porcelain chamber between two electrodes. In this way each pellet with its insulating powder covering constitutes a miniature cell corresponding to the larger and less numerous cells of the oxide-film arrester.

The autovalve arrester is another valve-type arrester suitable to all voltages and is offered by another manufacturer in preference to the aluminum arrester. It consists of a stack of carborundum disks separated by very thin insulating washers. A gap is used in series. The discharge spreads over the entire surface of the carborundum in the form of a glow, thus preventing localized heating and consequent arcing. As no arc is formed, the discharge is immediately interrupted when normal voltage is resumed.

A gap is generally used between the valve-type arrester and the line to prevent discharge with moderate voltage rise and to prevent leakage of current through the arrester. Where the gap is used to interrupt the discharge, it is usually provided with horns to lengthen and finally to extinguish the arc as it spreads upwards by its own heat.

Choke coils are used in the main-line conductors between the apparatus to be protected and the point where the arrester is connected.

The choke coil (Fig. 202) is essentially an air-core reactor. The impedance is small at normal frequency, but at impulse frequency the coil materially assists in directing the disturbance into the arrester. As the choke coil is in the main line and subject to system short-circuit currents, it must be mechanically braced to withstand the severe stresses.

Grounding.—The station grounding system performs three classes of service: (1) safeguarding of employees, (2) protection of circuits and apparatus, and (3) operation of relays. In all cases it is very important that a reliable, low-resistance, large-ampere-capacity ground connection be installed. Large buried bodies of steel or iron such as penstocks, abandoned steel sheeting, and water mains often form excellent grounds, but in some cases apparently well-grounded objects and structures are

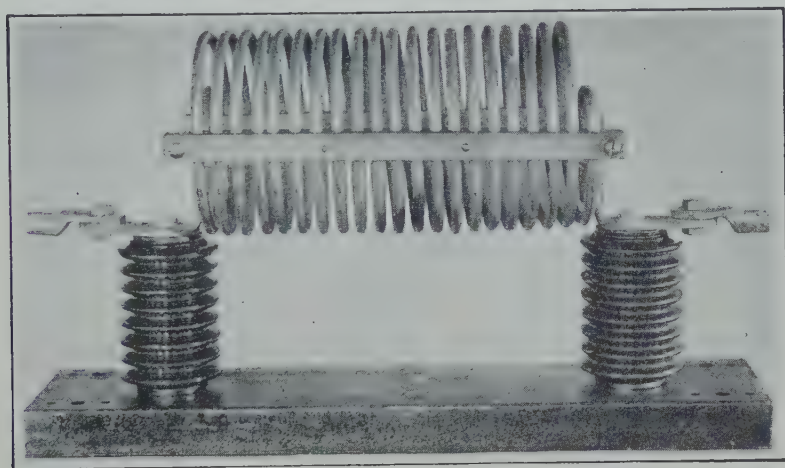


FIG. 202.—Choke coil for indoor use—Electric Power Equipment Corporation.

found by test to have a considerable resistance to ground. It is, therefore, always necessary to select carefully the location of the grounds and to test them carefully, not only when they are installed but periodically during their service.

A copper-plate ground is made by burying a plate of copper about 3 or 4 ft. square sufficiently deep in the earth to lie in permanent moisture. The iron-pipe ground is very easily installed and very effective. Galvanized-steel 1- to 1½-in. pipes without joints should be driven to permanent moisture. Pipe grounds should not be placed closer to each other than 6 ft. At least two pipes should be used for every installation and more are preferable. In some instances, it is advisable practically to surround the station with ground pipes.

Equipment to be grounded includes the following:

- Switchboard frames and supports.
- Instrument and meter cases.

Non-current-carrying metal parts of switchboard devices with which the attendant may come in contact.

Transformer frames and cases.

Oil circuit-breaker frames and mechanism.

Rheostats.

Generators, motors, and starters.

Instrument transformer secondaries, at the transformers.

Lighting transformer neutrals.

Steel supports, structures, grilles and fences.

Bases of disconnecting switches, and fuses, especially when mounted on masonry.

Lightning arresters. These should have individual grounds with straight direct-ground connections.

Energy-dissipating Rheostats.—Rheostats of large capacity are often desired for loading generators while under test and are sometimes installed permanently to regulate speed, ground the neutral, and for other purposes.

A submerged-wire rheostat consists of a number of coils of wire submerged in water which acts as a cooling medium. This type has been successfully used for voltages up to 500 volts. The coils may be arranged in a barrel or tank or may be submerged in the river. Adjustment of load may be obtained by the use of taps or interconnections.

The electrolytic rheostat consists of a group of metal plates or electrodes immersed in a solution or electrolyte which acts as a conducting medium. Such rheostats may be used with voltages up to 1000. Various electrodes such as lead, carbon, copper, or iron and electrolytes such as acids, bases, and salts may be used, provided the combination does not produce chemical action. Adjustment of load may be obtained by varying the distance between the electrodes, the depth of immersion, or the strength of the electrolyte.

A water rheostat consists of a set of terminals, usually plates or pipes, immersed in water which acts as a conducting medium. Water rheostats are adaptable to generator voltages above 1000. The terminals may be arranged in a tank but are more commonly placed directly in the river. Adjustment of load may be obtained by varying the spacing of terminals or the depth of immersion.

Outdoor Stations.—Where conditions are favorable, considerable advantage is gained by placing transformers and high-voltage switching equipment outdoors. The expense of outdoor supporting structures is less than that of enclosing buildings, and greater clearance can be obtained. However, in localities where ice and sleet frequently prevail, in extremely dry climates near the ocean where salt-laden air is apt to build up a cumulative conducting coating on the insulators, and in mountainous regions where level ground is not available, it is generally preferable to house the electrical equipment.

Transformers, breakers, and other high-voltage equipment are readily adapted to outdoor installation by using outdoor-type bushings. Buses and connections require somewhat greater clearances and higher insulation than when installed indoors. Control wiring must be carefully run in leaded cables and terminated in waterproof housings.

Structures for the outdoor station must withstand the stresses from the lines and the suspended buses, the wind and ice loads, and the dead weights of any suspended apparatus. Because of the inconvenience of painting the high-voltage structures, the members are generally made of galvanized steel. Foundations, adapted to soil conditions, must be provided for the bases of the structures and for supporting the transformers, breakers, arresters, and other large equipment. Lighting should, in general, be directed upward so as to light the equipment and avoid glare (see Fig. 287, Chap. XIII, and also frontispiece for arrangement of outdoor yards at Davis Bridge plant).

Automatic Stations.—The development of automatic and semiautomatic power-station and substation control has very greatly aided in reducing the personnel of the operating forces of the large power generating and distributing systems.

Small generating stations may be made entirely automatic, so that the generators will be automatically started, synchronized, and connected to the system and again disconnected and shut down, the operation being governed by load conditions or by water conditions as desired. Substations may also be used to start and synchronize the converters or motor generators automatically as dictated by the demands on the system.

Generating and substations are sometimes made semiautomatic, that is, the equipment is controlled and meters read by an operator from a distance. The development of various systems of supervisory control and metering has greatly assisted in the use of stations of this type by reducing to a minimum the number of wires required between the station and the operating point. The main equipment is practically the same as in the operated station, but the switchboard and control are manifestly different. In fact, the supervisory system simulates somewhat the wiring of a telephone switchboard, but in spite of its complicated nature it has proved to be thoroughly reliable.

STATION WIRING

Scheme of Connections.—The scheme of connections to be used deserves extensive study in the early stages of station design. The principal features the design should provide for are: (1) operation of generators and transformers at best efficiency under all conditions of station load by a suitable busing and grouping of equipment, (2) retirement of any machine, breaker, or circuit for inspection or repairs without

interfering with normal station output, (3) limitation of short-circuit current by suitable separation of groups of generators or if necessary by the use of reactors, (4) adequate relay protection of equipment, buses and circuits, and automatic isolation of defective parts without spreading the trouble, (5) facilities for testing any line with one of the generators and transformer banks without cutting the station apart, (6) reliable source of supply for the station auxiliaries, (7) suitable provision for metering and synchronizing. A low-voltage bus is necessary when feeders leave the station at generator voltage, otherwise its use is open to question. The double-bus scheme with selective oil circuit-breakers is generally desirable for either the low-voltage or the high-voltage bus. In cases of many feeders and few generators or *vice versa*, however, the group-bus scheme may prove more economical as each of the less important circuits can then be provided with but one breaker. The ring-bus and transfer-bus schemes offer many advantages and in some cases are preferable to the double bus. A typical double-bus scheme of connections is shown in Fig. 203 for the Caribou plant of the Great Western Power Company.

Main Wiring.—Where a main low-voltage bus is used, it is a very vital part of the station wiring, and a ground or short circuit on it would be very serious. The best possible design, construction, and protection should, therefore, be used. The conductor may consist of copper wire, cable, rod, tubing, or bars. For all but very small stations the low-voltage bus usually consists of copper tubing or bars mounted on porcelain bus supports and enclosed in a masonry structure. The joints and connections are very important. The gross-contact area should be at least ten times the cross-sectional area of the bar, and sufficient bolts should be used to assure a firm contact pressure. Clamps are satisfactory for securing the bus to the bus supports, but for current-carrying joints bolts are likely to distribute the pressure more uniformly, are more positive, and occupy less space.

The selection of suitable bus supports is very important. A wet flashover test of two and one-half times working voltage between phases should be specified. The short-circuit stress which a bus support will be called upon to withstand depends upon the value of short-circuit current, the spacing between buses, and the spacing between supports. Porcelain bus supports are stronger under compression than under any other direction of loading, and in cases where short-circuit stresses are very excessive, this advantage is sometimes utilized by placing porcelain supports on two or more sides of the bus so that all porcelain is in compression.

Structures for enclosing the main low-voltage buses, disconnecting switches, oil circuit-breakers, instrument transformers, and other related equipment are built of monolithic concrete, precast concrete slabs

RATINGS

Main generators, 22,223 kv-a. each
 Exciter, 600 kw. each
 Exciter motors, 900 hp. each
 Main transformer banks, 3-7500 kv-a.
 Exciter transformer banks, 3-550 kv-a.
 Station power transformer bank, 3-125 kv-a.
 Station light transformer, 1-125 kv-a.
 Local line reactors, 2% at 6000 kv-a.
 Future bus reactors, 10% at 22,223 kv-a.

SYMBOLS

	Generator or Motor
	Power transformer
	Current transformer
	Potential transformer
	Reactor
	Non-automatic oil circuit breaker
	Automatic oil circuit breaker
	Starting compensator
	Electrically operated non-automatic air brake circuit breaker
	Lightning arrester with horn gap
	Present installation
	Future installation

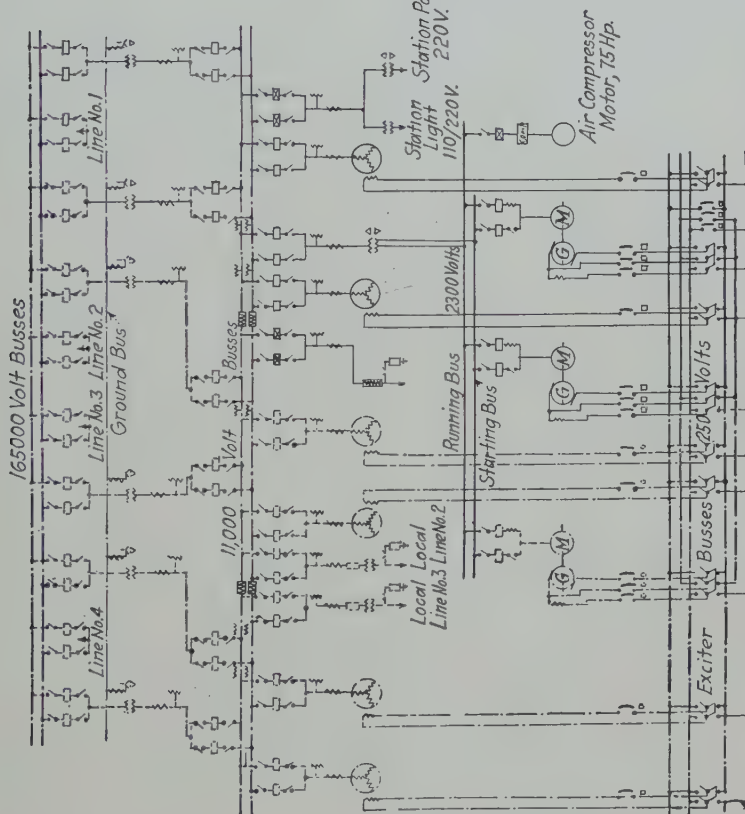


Fig. 203.—Scheme of connections—Caribou Power House of Great Western Power Company, California. (Courtesy of Stone and Webster Engineering Corporation.)

or blocks, brick, tile, sandstone, soapstone, slate, marble or asbestos lumber, or combinations of these materials. A form of construction which is usually satisfactory consists of a solid monolithic concrete main wall with barriers and shelves of precast concrete, brick, or tile, keyed into the main wall. Into the main wall can be cast the conduits and bolts or inserts for all bus supports, disconnecting switches, and other equipment. The barriers and shelves are thus relieved of mechanical loads and merely serve as isolating and protecting barriers. Doors or grilles should be provided at least 6 ft. high from the floor as a safeguard for employees.

When conductors carry more than 2000 amp., steel beams and reinforcing steel, particularly loops around single conductors, should be avoided, as such steel is likely to become heated by induced currents and crack the concrete. With voltages of over 10,000 volts, steel beams and reinforcement in the structures and adjacent floors and walls should be kept away from the conductors to prevent the spreading of an accidental flash to ground. If this cannot be done the steel should be carefully grounded so as to form a definite path to ground and thereby avoid stray currents.

With the high-voltage wiring, the problem is seldom one of carrying capacity and mechanical stresses as with the low-voltage wiring but almost entirely one of insulation. The buses and connections are usually made of bare copper cable, copper tubing, or steel pipe. The size is generally dictated by mechanical considerations.¹ The conductors can be supported by pin-type or post-type bus supports or by strings of suspension insulators. Barriers are seldom used between phases, but for indoor installations the various buses and circuits may be placed in individual rooms or cells with porcelain bushings where the conductors pass through the walls.

Control, Instrument, and Signal Wiring.—The control, instrument, and signal system wirings, being extremely complicated and vitally important, require perhaps more careful planning than any of the other systems, using the best insulated wires and cables, carefully installed. Most of this wiring terminates at the main switchboard, where serious congestion is likely to occur. Much trouble can be avoided by the use of a carefully designed control-cable terminal room below the switchboard room.² In the terminal room provision can be made for neatly and systematically making all necessary cross-connections. In some cases all instrument buses and fuses can be located there, thus greatly simplifying the wiring on the switchboard. Where a conduit room is not feasible, some of its

¹ MEES, C. A., "Bus-bar Data for Outdoor Switching Structures," *Elec. World*, Jan. 8, 1916, p. 86.

² HOPKINS, R. A., "Features of a Switchboard Terminal Room," *Elec. World*, Mar. 13, 1926, p. 561.

advantages can be gained by providing a large terminal box or trench behind the board into which the conduits are brought. Care should always be taken to anticipate and provide for future changes in the wiring.

Multiconductor cables are sometimes used to great advantage for the control and instrument wiring. They may be terminated at one end in suitable boxes in the control-cable terminal room and at the other end in similar boxes scattered throughout the station. Extensive grouping of control circuits should be avoided, however, and control and instrument wiring should not, as a rule, be placed in the same cable or conduit.

Wires and Cables.—The usual conductors used for power-station wiring are of soft-drawn, annealed copper. Standard sizes of bare conductors are given in Table 86, Chap. XI. It is sometimes advisable to use solid conductors up to No. $\frac{4}{0}$ in exposed work where stiffness is needed, but the majority of the wiring should be stranded. It is always advisable to use stranded conductors even in small sizes for important control circuits as they are less liable to be broken. Flexible stranding is advisable for cables larger than No. $\frac{4}{0}$ if pulled into conduits. Rope-core single-conductor cables are sometimes economical where the current exceeds 1000 amp. They are somewhat larger than standard cables. Hard-drawn or semihard-drawn copper is generally used for outdoor work where mechanical strength is important and the added stiffness is not objectionable (see Table 87, Chap. XI, for sizes and properties).

Insulation for electrical conductors usually consists of either rubber, cambric, or paper. Rubber insulation of National Electrical Code¹ quality and thickness (N.E.C. rubber) is used for practically all station wiring of sizes No. $\frac{4}{0}$ and smaller when carrying voltages of 600 volts and less. For higher voltages or where somewhat better quality is desired, 30 or 40 per cent Para or Hevea rubber is sometimes used. Varnished-cambric insulation of various thicknesses is used for practically all the main wiring of the station. Oil-paper insulation costs less than either rubber or varnished cambric, but cables so insulated are liable to be injured by being bent sharply. For this reason, paper-insulated cables are seldom used in power-station conduits but are almost universally used with lead sheaths for underground duct work.

Tapes and braids are placed over the insulation to protect it from mechanical injury. For conduit work a weatherproof braid is used and for exposed work a flameproof braid is preferable. For lighting circuits it is convenient to use white braid for the neutral wire to distinguish it from the other wires. For control, instruments, and signal wires, it is sometimes found convenient to make use of several colors of braids for identification.

¹ National Electrical Code, National Board of Fire Underwriters, New York.

Lead sheaths are used in general where wires or cables are exposed to moisture, oil, or other agents which would deteriorate the insulation.

TABLE 80.—ALLOWABLE CARRYING CAPACITIES OF WIRES

B. & S. gage	Diameter of solid wires, mils	Area in cir- cular mils	Table A, rubber in- sulation, amperes	Table B, varnished- cloth insula- tion, amperes	Table C, other insulation, amperes
18	40.3	1,624	3		5
16	50.8	2,583	6		10
14	64.1	4,107	15	18	20
12	80.8	6,530	20	25	25
10	101.9	10,380	25	30	30
8	128.5	16,510	35	40	50
6	162.0	26,250	50	60	70
5	181.9	33,100	55	65	80
4	204.3	41,740	70	85	90
3	229.4	52,630	80	95	100
2	257.6	66,370	90	110	125
1	289.3	83,690	100	120	150
0	325	105,500	125	150	200
00	364.8	133,100	150	180	225
000	409.6	167,800	175	210	275
0000	460	200,000	200	240	300
		211,600	225	270	325
		250,000	250	300	350
		300,000	275	330	400
		350,000	300	360	450
		400,000	325	390	500
		500,000	400	480	600
		600,000	450	540	680
		700,000	500	600	760
		800,000	550	660	840
		900,000	600	720	920
		1,000,000	650	780	1000
		1,100,000	690	830	1080
		1,200,000	730	880	1150
		1,300,000	770	920	1220
		1,400,000	810	970	1290
		1,500,000	850	1020	1360
		1,600,000	890	1070	1430
		1,700,000	930	1120	1490
		1,800,000	970	1160	1550
		1,900,000	1,010	1210	1610
		2,000,000	1,050	1260	1670

1 mil = 0.001 in.

From National Electrical Code.

End bells or potheads should always be used at the ends of lead-covered cables for voltages of 2000 or more.

The current-carrying capacity of a cable, where voltage drop is not the determining factor, is limited by the temperature which is considered safe for the insulation employed. The ratings given by the National Electrical Code, Table 80, are conservative for the smaller circuits of the station wiring. For the more important circuits, however, and particularly for the higher voltages, consideration should be given to (1) voltage, (2) number of conductors per cable, (3) number of cables in adjacent ducts, and (4) ambient temperature of surroundings.

Splices in cables are of great importance and deserve expert attention in order that the splices may be at least equal to any other part of the cable in conductivity, insulation, and protective covering. Fireproofing of certain exposed cables is sometimes advisable, as, for instance, at the main generator terminals. This consists of covering the cable with asbestos tapes and fireproof paints.

Conduits and Ducts.—Rigid iron conduit is used to enclose the large majority of the station wiring. It is manufactured from steel tubing, the interior being lined with flexible, insulating enamel, the outside either enameled, sherardized, electrogalvanized, or hot galvanized. Installation of the conduit must keep pace with the building structure, and, therefore, accurate and well-detailed conduit plans are of great importance. Galvanized or sherardized conduit is preferable to black enameled conduit for first-class installations. When conduits are installed in cinder concrete, the cinders should be carefully selected to avoid acidity and should be thoroughly washed. It is also advisable to protect the conduits by asphalt paint or cement grout.

Fiber conduit is lighter and less expensive than iron conduit and is, therefore, used in preference to iron quite generally for concealed work in sizes 2 in. and larger, but, on account of its comparative fragility, it should always be enclosed in concrete.

AUXILIARY POWER AND LIGHTING

Source of Supply.—The total amount of power required for station auxiliaries and lighting as a percentage of the station output will vary from $\frac{1}{2}$ to 5 per cent, depending on size of station, general type of hydraulic development, and many other factors. The source of supply is usually from the main bus through transformers, although with the larger stations auxiliary generators driven either by the main water wheels or by small individual wheels are often used. If auxiliary generators are used, it is advisable to provide at least one bank of transformers or some other standby source. Where the main bus is used as a source of supply, it is necessary to use feeder-voltage regulators on the lighting as the main bus voltage is not always constant.

Power Wiring and Equipment.—In providing for the auxiliary power load, the items to be considered should include excitation, turbine-room

cranes, intake-house cranes, gantry cranes, head gates, screen racks, sluice gates, locks, forebay ice prevention, transmission-line ice prevention, air compressors, lubricating-oil pumps, governor oil pumps, transformer oil and water pumps, filter presses, ventilating fans, generator-cooling fans, house-service pumps, sump pumps, electric heating, battery charging, and machine shop.

A carefully designed double-bus or sectionalized-ring bus system should be used, and circuits should be duplicated to important auxiliaries so that the retirement of any part of the system by accident or for purposes of maintenance will not curtail the station output. The main cables, being of large size, must be run in individual conduits. Such conduits must be of fiber or other non-magnetic material, as iron conduit surrounding a single conductor of an alternating-current circuit will become heated. The smaller circuits can be run with the three phases in the same conduit, and in this case iron conduit is preferred. It is always advisable, if possible, to run the conduits where they will be dry and thus avoid the use of lead-covered cable. If in some places leaded cables are necessary, they should have potheads if the voltage is 2000 or more, and in all cases the lead sheath should be grounded. Exposed wiring should have a flameproof covering over the insulation and should be substantially mounted on porcelain insulators. Solid wire is often desirable for exposed work, to secure neat appearance.

Motors and controllers should be carefully selected for the particular requirements of each service. The double squirrel-cage motor is designed to take 25 to 50 per cent less starting current than a standard motor and is, therefore, very satisfactory for station auxiliary drive. Slip-ring motors are sometimes required where adjustable speed or slow acceleration is desired. Push-button control for all motors is to be recommended, as the starting operations are simple and uniform, and generally a more convenient arrangement of equipment can be obtained.

Lighting Wiring and Equipment.—In providing for the station-lighting load, the items to be considered should include power station, outdoor switch yard, yard and grounds, storehouses, machine shops, dam, gate house, intake house, screen racks, locks, employes' cottages and buildings, display signs, and flood lighting.

The lighting-distribution system should, for convenience, consist of a separate circuit from the switchboard to each cabinet, but for reasons of economy it is usually necessary to group several cabinets on each feeder. Cabinets should generally be treated as fusing points only and located to best economic advantage from this standpoint, the switching being done by local wall switches. Three-way switching is essential wherever long rooms or passages are frequently entered from either end. Bus rooms, switch rooms, tunnels, and passages usually require this type of switching.

Emergency lighting is very essential. Three systems in common use are shown in Fig. 204. System C has the advantages that (1) the emergency lamps burn on the battery only during times when the normal source is dead, (2) no duplication of lamps is required at gages and other important points, and (3) outages are easily detected, as the emergency lamps form a part of the normal system. The battery should have capacity to supply the emergency lighting for a period of 1 hr.

Equipment, such as switches, receptacles, and lighting units should be rugged, convenient to operate, and accessible for maintenance. A large number of carefully located receptacles are of great assistance, both during construction and for maintenance. Water-tight equipment should

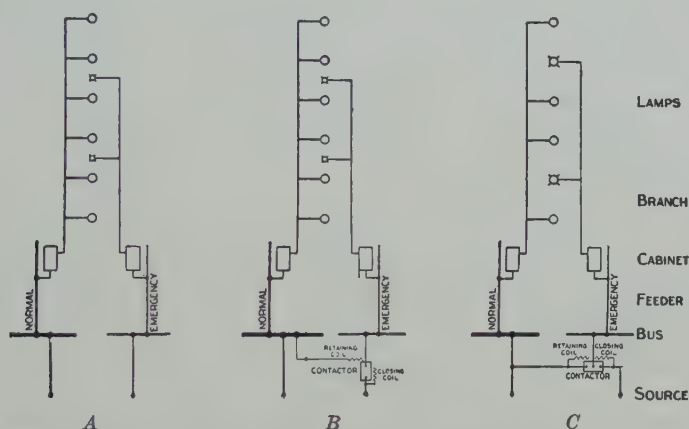


FIG. 204.—Diagrams of emergency distribution systems. *A*, emergency lamps always burn on emergency source; *B*, emergency lamps normally dark, but in emergency burn on emergency source; *C*, lamps normally burn on normal source, but in emergency burn on emergency source.

be used in all places subject to moisture. Guards should be used on all lights in passageways which are subject to breakage.

Illumination Design.—The common methods used in designing industrial lighting are applicable to such portions of the power station as the turbine room, switchboard room, offices, and similar open spaces. For the wheel pits, passageways, oil rooms, bus rooms, switch rooms, and other such parts of the building, however, careful attention should be given to side lighting, elimination of shadows, avoidance of glare, and individual location of lights with respect to machinery. It is often necessary in such places to use a large number of small lights advantageously located rather than a few large lights as would be preferable in an open space.

Glare and sharp shadows can be reduced by the use of enclosing globes of good diffusing glass. Ordinary ground glass and many kinds of opalescent glass are entirely inadequate as diffusers. Brilliancy can be reduced by using oversize enclosing globes.

TABLE 81.—ILLUMINATION-DESIGN DATA

Room	Normal intensity, foot-candles	Emergency intensity, foot-candles	Depreciation factor	Efficiency of utilization, per cent
Turbine room:				
Operation.....	4	2	1.3	40-50
Maintenance.....	8	2	1.3	40-50
Gate house.....	3	1	1.3	40-50
Wheel pits.....	3	1	1.4	20-30
Passages, tunnels.....	2	1	1.4	15-25
Oil rooms.....	4	1	1.5	25-35
Rheostats, reactors, transformers	4	1	1.4	40-50
Oil circuit breakers.....	4	1	1.4	35-45
Bus rooms.....	4	1	1.3	30-40
Battery room.....	4	1	1.4	25-35
Switchboard room:				
With ceiling lights.....	6	2	1.3	40-50
With glass ceiling.....	6	2	1.4	20-30
Offices.....	8	1	1.3	45-55

TABLE 82.—STANDARD-LAMP DATA

Bases are medium screw for sizes up to 200 watts and mogul screw for larger sizes. Standard lamps covered by this schedule are designed for burning in any position except that the 230-volt 750-watt and 230-volt 1000-watt lamps are designed for burning base up. These can be made for burning base down on request, in which case the light-center distance is reduced by $\frac{1}{2}$ in. Sizes 100 watts and smaller have inside frosted globes.

Volts	Watts	Initial lumens per watt	Dimensions in inches		
			Diameter	Length	Light center
115	25	8.9	$2\frac{3}{8}$	$3\frac{15}{16}$	$2\frac{1}{2}$
115	40	9.5	$2\frac{5}{8}$	$4\frac{7}{16}$	$2\frac{7}{8}$
115	50	10.8	$2\frac{5}{8}$	$4\frac{15}{16}$	$3\frac{3}{8}$
115	60	11.1	$2\frac{5}{8}$	$5\frac{1}{4}$	$3\frac{3}{4}$
115	100	13.2	$2\frac{7}{8}$	$6\frac{1}{16}$	$4\frac{3}{8}$
115	150	15.2	$3\frac{1}{8}$	$6\frac{15}{16}$	$5\frac{1}{4}$
115	200	16.1	$3\frac{3}{4}$	$8\frac{1}{8}$	6
115	300	17.4	$4\frac{3}{8}$	$9\frac{7}{16}$	7
115	500	18.8	5	$9\frac{13}{16}$	7
115	750	19.7	$6\frac{1}{2}$	$13\frac{1}{8}$	$9\frac{1}{2}$
115	1000	21.0	$6\frac{1}{2}$	$13\frac{1}{8}$	$9\frac{1}{2}$
230	100	10.6	$3\frac{1}{8}$	$6\frac{15}{16}$	$5\frac{1}{4}$
230	200	13.2	$3\frac{3}{4}$	$8\frac{1}{8}$	6
230	300	14.5	$4\frac{3}{8}$	$9\frac{7}{16}$	7
230	500	16.2	5	$9\frac{13}{16}$	7
230	750	17.5	$6\frac{1}{2}$	$13\frac{1}{8}$	$9\frac{1}{2}$
230	1000	18.2	$6\frac{1}{2}$	$13\frac{1}{8}$	$9\frac{1}{2}$

Large interiors, such as the turbine room and gate house, require high-capacity direct-lighting units on the ceiling above the crane. Prismatic reflectors are suitable for this purpose, as they direct most of the light efficiently to the working plane and at the same time provide some illumination for the upper part of the room. In the turbine room, the switching should provide at least two intensities. If desired for decorative effect, wall lights can be used in addition to the overhead lights, but they should be of low brilliancy and should not be expected to contribute appreciably to the illumination intensity.

Oil rooms should have gas-proof fittings with switches and receptacles either located outside or of gas-proof design. Battery rooms should have acid-resisting fittings. The use of light-colored acid-resisting paint in the battery room greatly improves the lighting.

Bus rooms and switch rooms having barriers require careful design in order to light the interiors of the compartments. Subcell disconnecting switches and ceiling disconnecting switches require particular attention. The use of white paint on the walls, ceiling, and barriers and the use of a relatively large number of highly diffusing lighting units greatly assist in obtaining satisfactory lighting.

The main switchboard room should have comfortable, attractive lighting, adequately providing for operation of the controls, reading of instruments, clerical work at the desk, and repair work behind the panels. Overhead units of the semiindirect type are suitable as they give a strong component of illumination on the vertical boards and minimize glare and shadows. They should be designed for distributing not over 10 per cent of their total output below the horizontal. The ceiling should have a dull-white finish. If these precautions are not regarded, disagreeable reflections are likely to occur on the faces of the instruments. Glass ceilings are sometimes provided and if properly designed give better results than windows in the walls. A skylight over the glass ceiling provides natural light by day, and lighting units between the ceiling and the skylight provide artificial lighting by night. The reflectors should be of glass if placed in the path of the natural light but may be of steel if placed at the sides. Prismatic sheet glass in the ceiling can be used to direct the light toward the switchboard, but this feature is of small importance and generally the more attractive rippled glass is preferred. Where a glass ceiling is used, it is very difficult to prevent reflections from instrument faces and this feature should have careful consideration.

The fundamental relation between lamp wattage and illumination intensity for a given area or room is expressed by the equation:

$$W = \frac{IAD}{EL}$$

where W = total wattage of lamps in watts.

I = illumination intensity in foot-candles. Suitable values are given in Table 81.

A = area of floor, square feet.

D = depreciation factor. Suitable values are given in Table 81.

E = efficiency of utilization depending on type of unit, shape of room, and character of interior finish. Suitable values are given in Table 81.

L = lamp efficiency in lumens per watt. Values are given in Table 82.

List of Textbooks and Handbooks on Electrical Engineering

JACKSON, D. C., and J. P. JACKSON, "Alternating Currents and Alternating-current Machinery," The Macmillan Company, New York.

LAWRENCE, R. R., "Principles of Alternating-current Machinery," McGraw-Hill Book Company, Inc., New York.

MAGNUSSON, C. E., "Alternating Currents," McGraw-Hill Book Company, Inc., New York.

RUSHMORE, D. B. and E. A. LOF, "Hydroelectric Power Stations," John Wiley & Sons, Inc., New York.

"Engineering Electricity," John Wiley & Sons, Inc., New York.

"Handbook for Electrical Engineers," John Wiley & Sons, Inc., New York.

"Standard Handbook for Electrical Engineers," 6th ed., McGraw-Hill Book Company, Inc., New York.

CHAPTER IX

PLANT ACCESSORIES

The main elements of a water power development—the dam, waterway, power house, and tailrace—require various detailed structures and equipment for use in operating the plant, for handling floods, etc., and these plant accessories will now be considered.

HEAD WORKS

The head works include the appliances for controlling the flow of water into the waterway and include the headgates, their operating mechanism and housing, the trash racks, etc.

Headgates.—The headgates may be constructed of wood, steel, cast iron, or steel filled with concrete, the type and size of gate depending upon the discharge capacity of the plant units. Large gates are usually of wood or structural steel, while gates for controlling flow into penstocks, if not too large, may often be of the cast-metal sluice-gate type.

The general design and arrangement of headgates should be such as to avoid unnecessary loss of head due to sudden changes in velocity. At the entrance to a canal, under normal conditions a number of gates must be used, with masonry piers between them. The upstream end of these piers should be rounded or formed as a cutwater to maintain smooth stream-flow lines. It is nearly always worth while to treat the downstream ends of the piers in similar fashion, to make a more gradual transition to the full canal section below the gates. Near both ends of the piers and abutments, there should always be vertical grooves for stop logs, so that the gates can be unwatered for repairs. Such stop logs have a thickness of 4 in. or more, depending upon the head to be carried. The grooves may be simply rectangular notches in the concrete or the notch may be lined with a steel channel or angles.

A single gate may be used for a penstock, if the latter is not too large. If two gates are required, rectangular openings with a concrete division wall or pier between may be built, gradually merging into the penstock section.

General dimensions of headgates for a number of plants, including velocity at the gate section, are given in Table 83, listed according to the type of development. It will be noted that in general a velocity at the gate section of from 0.05 to 0.10 $\sqrt{2gh}$ is common practice. (The relatively high velocity through the Turners Falls headgates was imposed by

TABLE 83.—HEADGATES—AREAS AND VELOCITIES

Plant	Water-way area, square feet	Gates			Maximum discharge, second-feet	Velocity, feet per second		H, feet	$\sqrt{2gH}$
		Number	Size, feet	Total area, square feet		In water-way	Through gates		
For canals (or flumes)									
Turners Falls.....	3,920 ^a	{ 9 8	{ 8.5 by 12 8 by 10 }	1,558	15,000	3.1	9.6	60	0.15
Garvins Falls.....	820	18	9½ by 15	2,560	12,000		4.7	60	0.08
Sherman Island.....	1,846	6	10 by 12	720	2,900	3.5	4.0	26	0.10
Verdi.....	152	10	9½ by 10	950	7,500	4.7	8.0	66	0.12
Hat Creek 1.....	232	8	5 by 8	320	500	3.3	1.6	92	0.02
Hat Creek 2.....	113	22	8 by 11	176	600	2.6	3.4	200 ±	0.03
		2	8 by 11	176	800	7.0	4.5	200 ±	0.04
For penstocks, conduits, or tunnels									
Searsburg.....	50	1	6 by 8	48	300	6.0	6.2	205	0.05
Davis Bridge.....	161	2	8 ft. circ.	100	1,770	11.0	17.7	350	0.12
New England Power Company 3.....	197	4	6 by 12	288	1,760	9.0	6.1	64	0.09
Bartlett's Ferry.....	352	2	16 by 20	640	4,080	11.5	6.4	112	0.07
Tallulah Falls.....		5	8 by 10	400	1,600	10.6	4.0	580	0.02
Fifteen Mile Falls.....	284	1 ^b	19 by 29.8	567	3,050	10.5	5.7	170	0.05
Sherman.....	133	1	14 ft. circ.	153	1,400	10.5	9.2	78	0.13
Wallenpaupack.....	153	1	16 by 20	320	1,750	11.4	5.4	330	0.04
Boulder Dam.....	700	1 ^c	10 high by 32 ft. circ.		9,300	17.7	12.9	500	0.07
Open wheel pit (or concrete flume)—concentrated fall									
Vischer Ferry.....		8	10 by 16	1,280	3,130		2.4	26.5	0.06
Vernon (new units)...		2	16 by 20	640	1,860		2.9	34	0.06
Amoskeag.....		12	10 by 20	2,400	5,100		2.1	46	0.04
Keokuk.....		60	8 by 22	10,500	47,000		4.5	35	0.10
Rainbow, Connecticut.....		2	{ 8 by 14 } { 12 by 14 }	280	2,200		7.8	56	0.13
McIndoes Falls.....		2 ^b	13.75 by 12	330	1,250		3.8	27	0.09
Conowingo.....		1 ^d	27 ft. circ. ^e	570	6,500		11.4	89	0.15
Safe Harbor.....		3 ^b	29.3 by 15.7	1,400	8,500		6.0	53	0.10
Rock Island.....		3 ^b	28 by 16.8	1,200	7,400		6.1	32	0.13

^a Eighteen head gates at lower end of canal near power house.^b Number of gates, etc., for one unit.^c For four units.^d At entrance to scroll case.

TABLE 84.—ALLOWABLE VELOCITIES AT GATE SECTION (MAXIMUM DISCHARGE)

Head, feet	Range of velocity, feet per second	
	From	To
25	2.0	4.0
50	2.8	5.6
100	4.0	8.0
200	5.7	11.3
300	7.0	14.0
500	9.0	18.0

the restricted location of an old gate house, which had to be used with some modification for an enlarged canal.)

Table 84 is based upon the values given in Table 83.

Wooden Gates.—Wooden gates, as commonly made, are ordinarily of horizontal oak or hard-pine timbers, held together with two or more vertical through bolts and usually with splined joints. The thickness

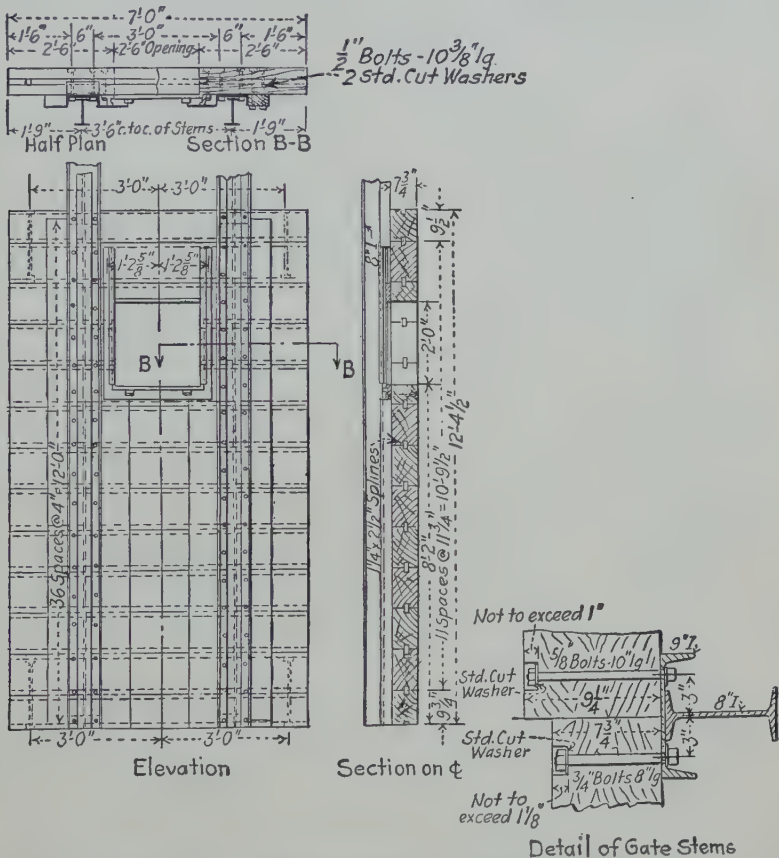


FIG. 205.—Details of wooden headgates—Plants 2, 3 and 4, New England Power Company.

of timber is from 4 in. upward, depending upon the head to be carried and size and shape of gate. The latter is usually rectangular with long dimension vertical.

A gate stem, one for small gates but more often two, extends for the height of the gate, to which it is securely bolted, and to a sufficient distance above the top of the gate to engage the lifting mechanism. The gate stem may be of timber with an iron rack, or with heavy gates rolled-steel sections may be used.

Details of a wooden gate for an opening 6 ft. wide by 12 ft. high in use at plants 2, 3, and 4 of the New England Power Company near

Shelburne Falls, Mass., are shown in Fig. 205. In this case two 8-in. steel I's are used for stems, each backed by a 9-in. channel, and each gate timber is bolted to this channel as shown. Note also the two additional vertical timbers to which the gate timbers are spiked and bolted. A filler gate about 2 ft. square is set in the gate about one-third the way down, for use in equalizing pressure by filling the penstock or canal while the turbine gates are still closed, thus requiring only the weight of the gate to be lifted. In this case the gate stem, of steel, takes the place of through bolts.

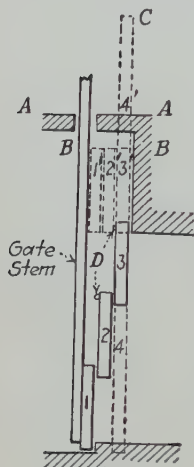


FIG. 206.

Multiple-leaf wooden gates are sometimes used to lessen the headroom required for the gate and gate stem and the capacity required for the hoist. The arrangement of such a gate with three leaves is shown diagrammatically in Fig. 206. The position of the leaves, with gate closed, is shown in full lines by 1, 2, 3; when open by 1', 2', 3'. The top of the gate when fully lifted or open is at BB, and the operating mechanism can be set at about level AA.

With a single-leaf gate for the same gate opening (shown dotted in Fig. 206, position 4 closed and 4' open), the top of the gate when open will be at C and the operating mechanism must be at a somewhat higher level.

The individual leaves of the multiple-leaf gate have their own grooves, and closely fitting wooden sealing strips are required between the leaves. A stop or metal knob (D, Fig. 206) causes the second and third leaves to start to open at the proper time.

Iron and Steel Gates. Sluice Gates.—These may be either circular, square, or rectangular and are usually constructed with a ribbed cast-iron body arranged to slide vertically in cast-iron side frames, the bearing surface of gate, gate seat, and side frames being of bronze to avoid rusting and consequent sticking of the gate. Adjustable bronze side or bottom wedges are often used on gate and frame, so set as to hold the closed gate in tight contact with the seat. Sluice gates are available for sizes up to about 10 ft. in diameter for circular gates and 10 ft. square or 9 by 12 ft. for rectangular gates. They deteriorate much less rapidly than wooden gates but are more expensive.

The 6- by 8-ft. iron sluice gates and hydraulic cylinder control as used at the Searsburg plant of the New England Power Company are shown in Fig. 207.

Wicket gates (also called flutter gates and butterfly gates) are used as headgates for penstocks and often near the wheels as well, to permit their inspection without unwatering the whole penstock line. The gate

is a circular disk mounted on either a vertical or horizontal shaft running through the penstock and operated by worm and gear, either manually or by motor or hydraulic control. When seated, the valve is not quite normal to the pipe axis; the valve edges are slightly beveled so as to insure a tight fit on the valve seat. Adjustable bronze seal rings are also sometimes used for this purpose. These gates are made of either cast iron or steel, ribbed for strength, and in extremely large sizes of structural steel. They are practically balanced in respect to pressure

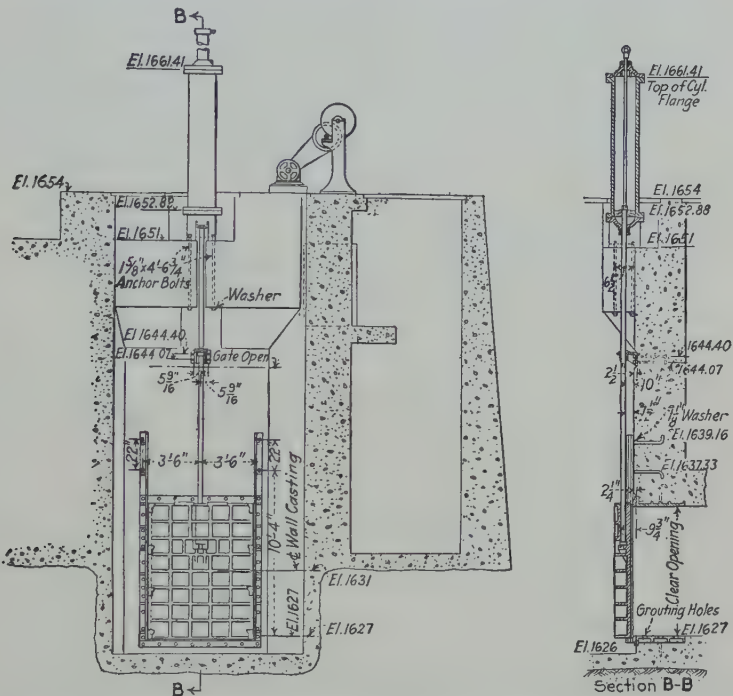


FIG. 207.—Six-by-eight-foot iron sluice gates and controls—Searsburg plant, New England Power Company.

and can be opened against pressure or closed against velocity without difficulty by a suitable operating mechanism. In some plants it is the custom, however, to close the turbine gates before operating the penstock valve. Sizes up to about 23 ft. in diameter are in common use, and the gates of this type at the Conowingo development are 28 ft. in diameter. Wicket gates cost much less than gate valves of the ordinary water-works type and are more convenient to operate.

A recent development of this type of gate by E. A. Dow, called the disk-arm valve, is shown in Fig. 208, as used at the Davis Bridge plant. This may be motor operated as shown or operated by pressure cylinder and may be arranged for operation either from the power-house switch-

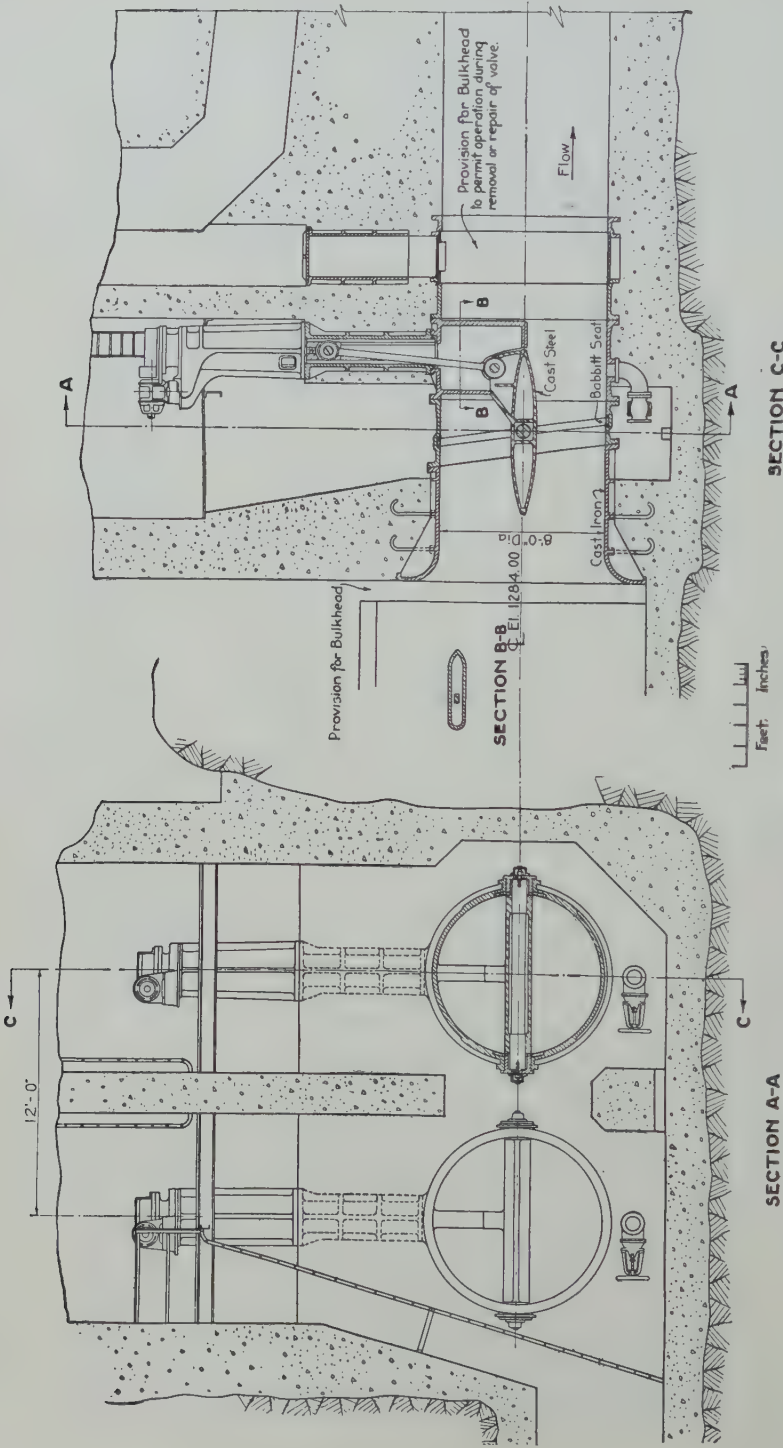


FIG. 208.—Dow disk-arm valve at Davis Bridge plant of New England Power Company.

board or at the valve. Note also the sealing ring of babbitt metal in the gate seat. This valve, by virtue of the closing force being applied at the bottom part of the disk, gives a very tight closure.

Needle valves of the Johnson type are balanced valves for penstock use, sometimes for headgates, but usually for control near the wheels. The arrangement of the type-A Johnson valve is shown in Fig. 209, for both closed and open positions. The plunger or valve is actuated by water pressure from the penstock line within either chamber *A* or *B*. To close the valve, chamber *A* is subjected to water pressure and *B* connected with the air; to open the valve, the operation is reversed.

The outlet diameter of the valve is commonly the same as the inlet diameter of the turbine casing. The rated diameter of the valve is that at the valve inlet. To avoid erratic movement of the plunger due to the pressure of air in chamber *A*, an air vent in the top of the internal cylinder

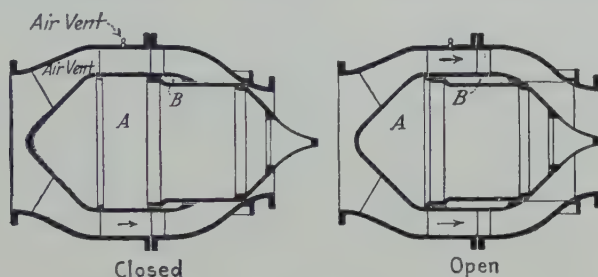


FIG. 209.—Johnson penstock valve, type A.

is provided as shown. An air valve is also required at the top of the valve body to permit the venting of air, when the penstock is being refilled after draining.

Various types of control for this valve are available, which may be manual or hydraulic.¹ The type-H control is automatic as to rate of opening and closing the valve, and damps the end of the closing stroke to prevent water hammer. This may also be under electrical control from a remote station in case of emergency. More commonly local hand operation is used, with opening and closing characteristics under the control of the operator.

The Johnson valve has the advantage of slight loss of head, as a relatively full water area is provided entirely around the valve. It is especially adapted for very large penstocks and has been built in sizes up to 21 ft. (inlet) by 14 ft. (outlet) diameter. These large valves weigh about 307 tons and are installed in the penstocks leading to the 70,000-hp. wheel units of the Niagara Falls Power Company.

Steel gates, built up of structural shapes, and often filled or partly filled with concrete, are used for gates of large dimensions. A recent

¹ *Eng. Jour.*, Engineering Institute of Canada, July, 1924, pp. 398-400.

example of steel headgate, 16 ft. wide by 20 ft. high, is found at the Bartlett's Ferry, Georgia, plant.¹ These are built up to a thickness of about 2 ft., and the lower compartment, about 2 ft. high, is filled with concrete. Counterweights made of concrete blocks, containing scrap

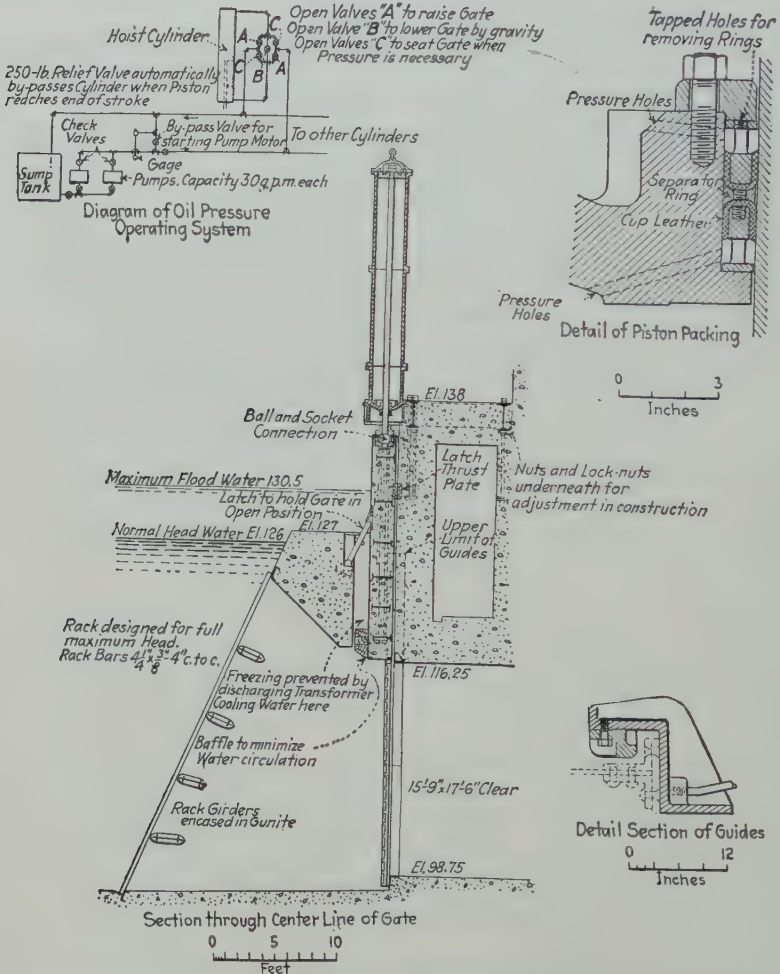


FIG. 210.—Concrete and steel headgates and racks—Vernon plant, New England Power Company.

steel, are placed in the other compartments. The "skin plate" is $1\frac{1}{16}$ in. thick and the crossbeams are 24-in. 73½-lb. I's spaced 2 ft. 1 in. at bottom, the spacing gradually increasing to 2 ft. 11½ in. at the top. A pressure-feed lubricating device for the gate guides is provided, together with a compressed-air system to keep the gate seats free from silt.

¹ Jour. Boston Soc. Civil Eng., March, 1926, pp. 93-125.

A steel gate entirely filled with concrete and operated hydraulically, as used at the Vernon station of the New England Power Company, is shown in Fig. 210. These cover an opening 15 ft. 9 in. by 17 ft. 6 in., and two gates are used for each wheel unit. Oil pressure at 200 lb. per square inch is used to operate the piston. A 2- by 3-ft. filler gate is used for each unit. Interference with closure due to freezing is entirely eliminated by piping the slightly warm discharge from the water-cooled transformers into the gate wells, which are planked over to retain the heat better. Further details of gates and operating mechanism appear on Fig. 210.

To make operation easier, large steel gates are sometimes provided with roller or wheel bearings. Thus, at the Mitchell dam¹ the gates are of the fixed-roller type 20 by 13 ft., with head on bottom of gate of 45 ft. The seat is bronze on cast iron. The wheel track is of cast iron with depressions for dropping the gate in sealing position. Wheels are so placed that only one at a time is over a depression until the gate is in sealing position; then all wheels drop in at the same time. The seal is then the same as for a sliding gate. A by-pass valve is provided for equalizing the pressure on the gate for raising, by means of a traveling crane, and a latching device to hold the gate is also provided. A gate complete, exclusive of the lifting device, weighs 14.5 tons.

The Stoney gate (invented by F. G. M. Stoney, London, in 1883),² of the separate-roller train type, has been used for openings as large as about 40 by 50 ft. and for lesser depths up to widths of 90 ft. The gate moves in vertical grooves in masonry piers on trains of rollers which are independent of gate and pier. The rollers have their bearings in two vertical side bars, at the top of which a loose sheave is placed. A chain or wire rope passes around their sheave and has one end attached to the gate and the other end fastened to some fixed point on the service bridge connecting the piers. Thus the train of rollers, usually counterbalanced by weights, is made to move half as fast as the gate, compensating for the gates moving on the diameter of the rollers while the rollers move on their radii. There is no pressure carried to or through the axles of the rollers. The pressure passes directly across the diameter of the roller from one side of the periphery to the other. Each gate is generally suspended through its center of gravity by link chains or by two or more screw rods from the service bridge. The upper ends engage with nuts (with antifriction bearings) which are turned by powerful gearing to raise and lower the gate, being aided by the counterweights, usually about three-fourths the weight of the gate.

¹ See *Report of Hydraulic Power Committee, Nat. Electric Light Assoc.*, pp. 58-63, 1924.

² See WEGMANN, "Construction of Dams," 6th ed., pp. 351-357, 1911; also *Report of Hydraulic Power Committee, Nat. Electric Light Assoc.*, pp. 64-70, 1924.

With this type of gate, friction is so minimized that gates subjected to as high as 400 tons of water pressure can readily be moved by hand.

Many such gates are in use abroad and in a number of installations in this country, the earlier ones on the canals at and near the Great Lakes and more recently at power developments in the South.

Details of the rollers and arrangement of bronze sealing rods, etc., of the 26- by 50-ft. Stoney gate at the La Gabelle plant of the Shawinigan Power Company are shown in Fig. 211. These gates are normally motor operated but may also be hand operated.

Broome Caterpillar Gates.—Gates built under the Broome patent have been used at a number of plants. Those at the Turners Falls devel-

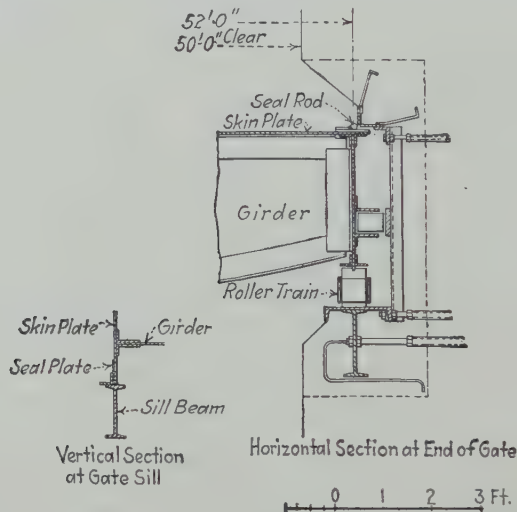


FIG. 211.—Stoney gates—La Gabelle Plant, roller and gate details.

opment are installed at the end of a long canal at the entrance to the power house. Each of the six units has three gates, each gate covering a clear opening about 9.5 ft. wide by 15 ft. high, raised by a motor-operated gantry crane (see Fig. 217). A 2- by 4-ft. filler gate with by-pass through the gate piers is used with the Broome gates.

The Broome gate is built with the seat inclined to the guides in such a way that the water pressure on the upstream side of the gate has a component in the direction of closure, so that the gate is self-closing provided friction is low. An additional advantage of the inclined seat is that it nearly eliminates sliding friction in opening, as the first motion of the gate tends to lift it from its seat. In the caterpillar design, frictional resistance is largely eliminated by the use of an endless chain with rollers running between tracks on the gate and guides upon which the gate runs. Hence, a hoist of the tension type can be used, as the gate need not be forced down in closing.

The intake arrangement at the Rainbow, Conn., plant, constructed in 1925, is shown in Fig. 212. The intake arrangement originally considered for the Rainbow plant called for two caterpillar gates each 14 ft. high by 10 ft. wide for each unit. Further study indicated, however, that the cost could be greatly reduced by the use of one sliding gate 14 ft. high by 12 ft. wide and one caterpillar gate 14 ft. high by 8 ft. wide. The combined area is the same, and the sliding gate will be weighted with concrete so that it will be heavy enough to close by its own weight under full turbine discharge, with the caterpillar gate open.

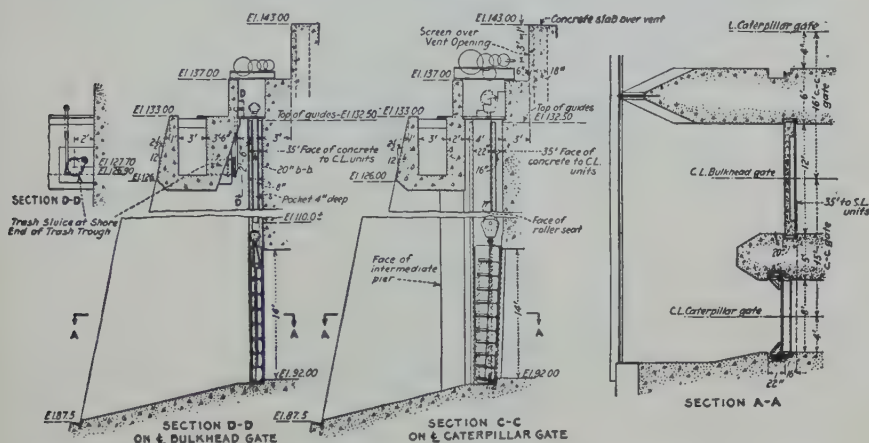


FIG. 212.—Headgates at Rainbow development on Farmington River, Connecticut.

In other words, it will slide against the friction resulting from a head difference sufficient to put the entire flow through the smaller gate. When the sliding gate is down, the caterpillar gate will be closed under full head, and of course will be opened first when it is desired to fill the scroll. This installation costs about two-thirds as much as the design using the two caterpillar gates and is equally good, the only requirement being that the sliding gate be operated first in case it is necessary to close the headgates with the turbine gates open.

Gate Hoists.—Gate-hoisting devices may be (1) for mechanical operation, hand or motor operated, and often arranged to permit either method of operation; (2) for hydraulic operation; (3) for operation by a crane.

Mechanically Operated Hoists.—A common arrangement for ordinary gates is shown in Fig. 213. A heavy spur pinion engages with the rack segment, which is usually of cast iron, bolted to the gate stem. The pinion and roll shafts are held firmly in position by journals cast with heavy bedplates bolted to a wooden frame. The hoist is operated by a level which fits socket holes in a wheel attached with ratchet and pawl to hold the gate as it is being lifted. For single-stem gates this type of

hoist has a maximum capacity of about $2\frac{1}{2}$ tons; for double-stem gates about 5 tons.

For larger gates the worm- and gear-wheel arrangement (Fig. 214) is used with dimensions and capacities as given in the accompanying table for either single- or double-stem gates.

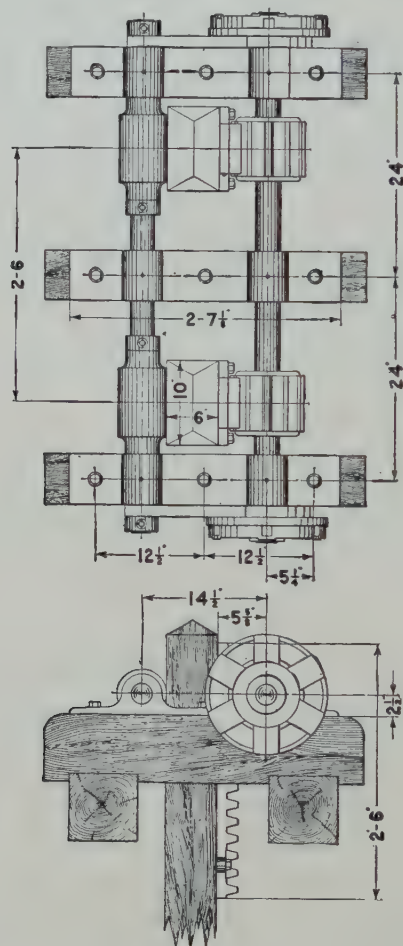


FIG. 213.—Gate hoist, type AA, S. Morgan Smith Company. Capacity 5 tons (two stems).

A geared hoist arranged for crank operation is also common (Fig. 215), with maximum capacity of $10\frac{1}{2}$ and $5\frac{1}{4}$ tons for double- and single-stem hoists, respectively.

A hoist for large gates ($7\frac{1}{2}$ to 30 tons lifting capacity), motor operated, is shown in Fig. 216. Such a hoist may also be arranged for auxiliary hand operation.

Hydraulic operation for large gates is quite common. Some of the details of the hydraulic control as used at the Vernon plant of the New

England Power Company are shown in Fig. 210, page 484. The hydraulic cylinder is 28 in. in diameter, and oil under a pressure of about 200 lb. per square inch is used, giving a lift of about 60 tons. The pumps are of the triplex-plunger type, 2 by 6 in., on the same shaft, operated by a 10-hp. motor at 220 volts.

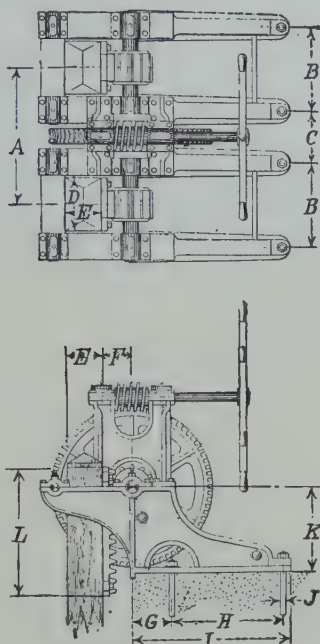


FIG. 214.

DIMENSIONS OF STYLE "EE" "FF" AND "LL" GATE HOISTS (*S. Morgan Smith Co.*)

	A	B	C	D	E	F	G	H	I	J	K	L	Maximum Capacity
EE	2 6 $\frac{3}{4}$	19	11 $\frac{3}{4}$	12	8	6 $\frac{1}{2}$	8 $\frac{3}{4}$	2 1	2 11 $\frac{1}{2}$	1 $\frac{3}{4}$	1 7 $\frac{1}{4}$	2 5 $\frac{1}{4}$	12,000 lb.
FF	2 6	13 $\frac{1}{6}$	18 $\frac{5}{8}$	8	8	5 $\frac{5}{8}$	4 $\frac{9}{16}$	1 2	2 0	1 $\frac{1}{4}$	1 3 $\frac{1}{2}$	2 11	56,000 lb.
LL	3 6	20	22	10	8	8 $\frac{3}{8}$	7	3 3 $\frac{1}{2}$	3 2 $\frac{1}{2}$	1 $\frac{1}{2}$	2 3	3 0	21,000 lb.

The gate is lifted hydraulically and closes by gravity action but may be seated by pressure when desired.

Crane operation or lifting for large gates may in some cases be best adapted, particularly where there are a considerable number of large gates.

At the Turners Falls plant, the gantry crane used for gate lifting is shown in Fig. 217 (also see Fig. 298, Chap. XIII). The gates are of the Broome type and are described on page 486. In Fig. 217 a gate is shown,

lifted somewhat above the full open position, the crane hook being inserted in the gate eyebar. Ordinarily an additional lifting link or steel eyebar about 16 ft. long is used between the crane hook and the gate eyebar. The crane is motor operated and of sufficient capacity to lift the full weight of the gate under pressure, without allowance for the roller bearings.

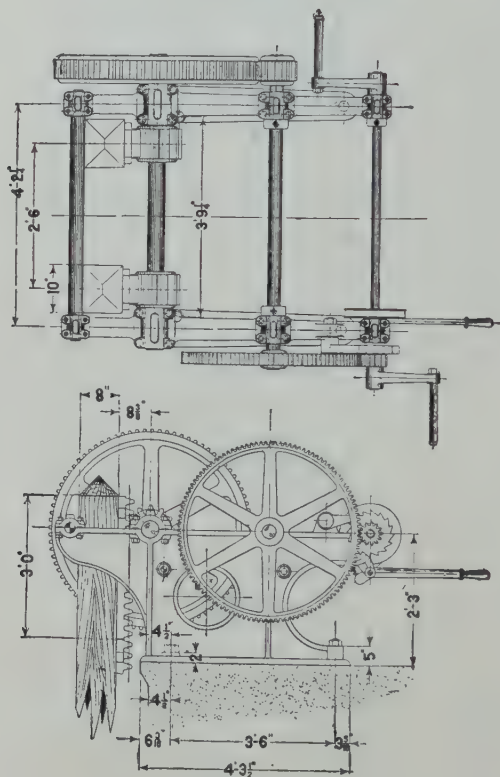


FIG. 215.—Gate hoist, type KK, S. Morgan Smith Company. Capacity 10.5 tons (two stems).

Racks.—Racks (also called trash racks and sometimes screens) to intercept material carried by the water which may not be readily passed through the turbines are commonly placed above the gate house at the dam. In the case of a long canal, another set of racks may also be placed near its lower end. As commonly constructed, racks are of relatively thin flat steel bars, with edge upstream and spaced from 1 to 3 in. apart or more in the clear. To facilitate raking or cleaning, the plane of the face of the racks is often inclined away from the vertical at an angle of about 15 to 20 deg., with upper end farthest downstream. The rack bars are spaced and held in place by cross-tie rods and “spools” or spacers

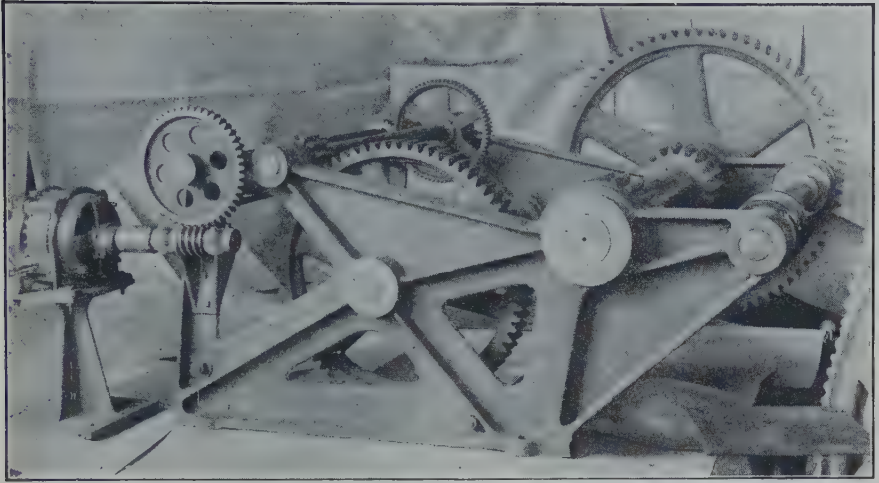


FIG. 216.—Motor-operated gate hoist—Dilts Machine Works, Fulton, N. Y. Capacity 7.5 to 30 tons.

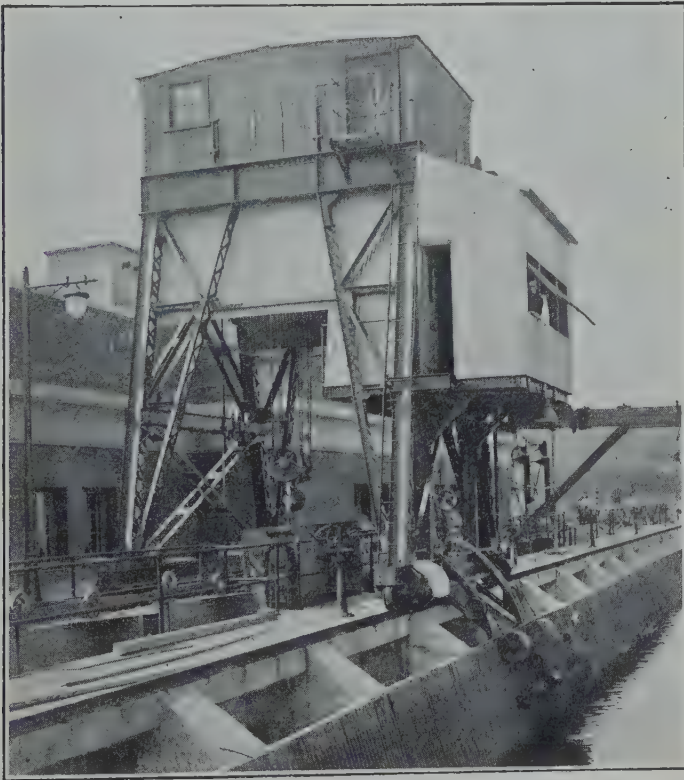


FIG. 217.—Gantry crane and mechanical rack rake—Turners Falls development. (Courtesy Turners Falls Power and Electric Company.)

stream from the center of the bars to permit easy use of a rake in cleaning.

The total net rack area for waterway should be such that undue loss of head will not result. This loss of head is due (1) to sudden contraction at the rack section and to (2) sudden enlargement beyond the rack section. * The allowable velocity through the net rack section to avoid too great a loss of head will vary from about 1 to 3 ft. per second, depending somewhat on the tendency to clog.

In Fig. 218 details of the racks and rack supports at the Searsburg plant of the New England Power Company are shown. Note that these rack bars have rounded corners, largely eliminating loss of head, and are built in sections about 21 ft. 11 in. long by 2 ft. 11 in. wide with alternate sections fixed and removable. The clear width between rack bars is $1\frac{1}{4}$ in. The bars are 3 by $\frac{1}{4}$ in. except the end bars of each section, which are 3 by $\frac{3}{8}$ in. For these racks at full-gate wheel capacity (300 sec.-ft.):

Gross area at racks = 380 sq. ft.; $V = 0.79$

Net area at racks = 270 sq. ft.; $V = 1.11$

The proper interval in the clear between rack bars is an important detail. If too small—1 in. or less—the racks will frequently clog with twigs and leaves and require an excessive amount of cleaning. If too large, material will pass which may clog or injure the wheel runner. Obviously the maximum allowable size of opening will depend somewhat on the size of wheel passages and the type of wheel. Spacing of 2 to 3 in. on centers of rack bars is common, and the general tendency is toward a wider spacing than that formerly in use.

The propeller type of wheel, particularly the Nagler runner with its few blades, permits much wider rack-bar spacing than the ordinary

FIG. 218.—(Continued.)

NOTES AND SPECIFICATIONS

- All material M.O.H. structural steel except as noted.
- All rivets and bolts $\frac{3}{4}$ " ϕ , open holes $1\frac{1}{16}$ " ϕ .
- Holes to be shop punched in beam webs for field fastening of wire mesh, 3 rows in G_1 , and 2 rows in B_1 , B_2 , and B_3 ; holes to be 24" centers.
- No bearing plates required.
- Paint all surfaces two shop coats red lead and linseed oil.
- All field connections bolted.
- Out to out width of rack bar sections must be exact necessitating pipe separators to be cut to exact length and to have square ends.
- Detailed shop drawings to be furnished for P. C. Co. approval.
- An opportunity for P. C. Co. inspection of all material after fabrication and assembly must be afforded before shipment.
- Strength of all beams and connections to be such as to carry a water head of 27 ft., working point to water surface, assuming that all the racks are completely clogged with debris and the following unit stresses are not to be exceeded:

Bending on extreme fibers $27000-420 \frac{L}{B}$

Shear on beam webs 12,000 #/□"

Bearing of rivets and bolts 24,000 #/□"

Shear on rivets and bolts 12,000 #/□"

Bearing of steel on concrete 500 #/□"

The rods $\frac{3}{4}$ " ϕ 2' 10" long, threaded $1\frac{1}{8}$ " each end.

Separators for tie rods, 1" ϕ extra strong wrought iron pipe $1\frac{1}{4}$ " long (exact).

Separators for rivets, 1" ϕ extra strong wrought iron pipe $1\frac{1}{8}$ " long (exact).

reaction wheel, as relatively large material will pass through this type of wheel without difficulty.

The design of the rack crossbeams and supports is usually made upon the basis of full hydraulic pressure occasioned by possible clogging such that for a short time, anyway, little or no water is passing through. While this possibility usually exists, it is remote, and, hence, higher unit stresses in the steel beams may be used than usual. (Note the basis used in the design of the Searsburg racks, Fig. 218.)

The top of the racks should be placed a little above high-water level in the canal or gate house, and usually it is advantageous to have a trough just downstream from the racks into which material raked from the racks can be dumped. Provision can usually be made at the river end of the gate house in ordinary-sized plants for a pit in the side walls into which waste material may be dumped, from which it may be occasionally flushed into the river by means of a small gate opening into the head water near the racks.

For large plants a continuous trough back of the racks may be used as shown in Fig. 217 at the Turners Falls plant. This is occasionally flushed out, through a gate-controlled sluiceway, to the river below the power house. In this case, as will be noted, a mechanical raking device is used as a part of the gantry-crane equipment operated by the crane motor. This mechanical rack rake has been found very satisfactory and is well adapted for use in a station of this size, where the wheel runners and racks have openings large enough to permit ordinary trash to pass, the racks only collecting large debris which could not be handled with other than some slow, powerful mechanism of this character.

Another unusual feature at this plant is the setting of the rack bars at a slight angle from vertical upstream planes to accord better with the direction of flow to the racks, which is not quite normal, due to a bend just at the end of the canal.

During 1926 a mechanical rack rake, constructed by the Newport News Shipbuilding and Dry Dock Company, was installed and has since been in use at the Vernon plant of the New England Power Company. Similar rack rakes have been installed and are in use at the four new plants of the company built during the period 1926-1931, on the Connecticut and Deerfield rivers.

Additional data in regard to racks and settings will be found for the plants listed in Table 75, Chap. VII.

*Ice Formation at Racks.*¹—In northern latitudes the possibility of clogging of racks by ice and consequent interruption of plant service must be given careful attention and thought in the design of the plant.

Sheet ice commonly causes no difficulty in plant operation other than to diminish the discharge capacity of the canal when it forms, and

¹ Report of Hydraulic Power Committee, Nat. Electric Light Assoc., pp. 47-52, 1922.

occasionally, in some locations, ice jams, when sheet ice breaks up, may be a source of trouble. The clogging of racks, however, and sometimes of waterways and wheels as well, is due to needle or anchor ice which may be floating in the water.

Needle ice or "frazil" (so called from the French word signifying "forge cinders," suggested by its dull slushy appearance) forms in quick water, cold enough to freeze but too turbulent to permit the formation of an ice sheet. Under favorable conditions needle ice may form in large quantities, float downstream, even underneath sheet ice, and accumulate in such a manner that much of it may reach the racks at the plant.

Anchor ice may be formed when stones or other objects in a stream are cooled by radiation below 32°F. When relative loss by radiation decreases, as for instance on a cloudy day, it may become detached and rise to the surface, floating along like needle ice. It never forms under sheet ice.

The absence of sheet ice in portions of the river is thus the primary cause of needle- and anchor-ice formation. As a river becomes more completely developed, approaching a series of ponds, which favor sheet-ice formation, ice troubles, therefore, tend to diminish.

While needle and frazil ice are of a sticky consistency at their usual temperature, it is also true that a very slight increase in temperature of racks and water passages (a few thousandths of a degree is enough) will permit free passage. In many cases, therefore, the difficulty is solved by a heating plant, usually a steam boiler, which is used when necessary. Normally the time of trouble is short and confined to the few days just as the river is becoming frozen over in early winter.

The housing of the racks and gates is also important, and a heated gate house will decrease tendency to trouble.

Where the wheel units are of good size, a method of operation now commonly used is to arrange a part or all of the racks so that they can be lifted and allow the ice to pass through the wheels. As the run of ice is usually only for a few hours at a time, there will usually be enough heat stored in the concrete scroll cases to prevent the ice sticking. The crown plates, shafts, and guide-vane stems have also, as a rule, sufficient contact with the warm air of the power house, and enough heat is transmitted through them to prevent ice sticking.

The freezing of large sluice gates so that they cannot be opened is also a possible difficulty. To obviate this, it is now common to lag the gates and install steam or electric heaters between the lagging and the skin plate, which effectively prevents the formation of ice on the gate. Headgates have also been kept free from ice by compressed air emitted from pipes laid in front of the gates at sill level, thus utilizing the slightly warmer water near the river bottom.¹

¹ See "Ice Problems in Hydroelectric Plants," *Report of Hydraulic Power Commission*, Nat. Electric Light Assoc., 1930.

Gate House.—It will often be desirable to house the head or control gates. The size and dimensions of the structure required will depend upon the number, size, and type of gates to be used and the nature and arrangement of the hoists or controls. Usually a long, relatively narrow building will be required, with sufficient height to permit the raising of the gates and, if necessary, their removal.

Wooden-frame construction is frequently used for the gate house. For a more permanent structure, brick or concrete walls with tar-and-gravel, slate, or tile roof may be used. A suitable arrangement of windows for lighting and service doors for access and removal of equip-

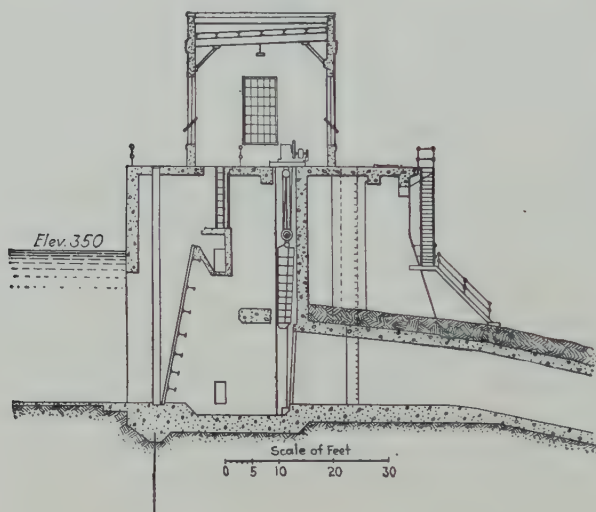


FIG. 219.—Gate house and equipment—Sherman Island plant.

ment should be planned, harmonizing the type of construction with the other features of the development, particularly those which are near by.

It will not usually be necessary to provide a crane or lift in the gate house, but care should be taken in design to provide suitable crossbeams or at least supports for temporary beams for the use of blocks and tackle in erection and in the future handling of gates or hoisting equipment.

The gate house for the Sherman Island plant, New York,¹ is shown in Fig. 219. This is of brick about 195 by 25 by 25 ft. high with concrete substructure and houses 15 motor-operated Broome gates, each 10 by 13 ft., weighing 25 tons. Note also the arrangement of racks and rack platform, rack wasteway, etc. The gates at this plant are novel in one respect. They have steel frames, concrete filled, and are built in three sections. The lower and middle sections are joined by short links with the joints calked; the upper and middle sections are connected with

¹ *Trans. A.S.C.E.*, vol. 88, pp. 1257-1328, 1925.

slotted links which permit the joints to open 12 in. in lifting. Water can thus enter the canal and relieve the pressure on the gate and facilitate raising.

Head Works for Reservoirs.—The arrangement of head works and gates at reservoir outlets has to be modified somewhat to provide for fluctuations in water level, which may be many feet in the case of a large reservoir. If an open channel or canal is located at the reservoir outlet, the gates must be placed sufficiently low in elevation to permit flow at low-reservoir level. This means that for all higher levels of

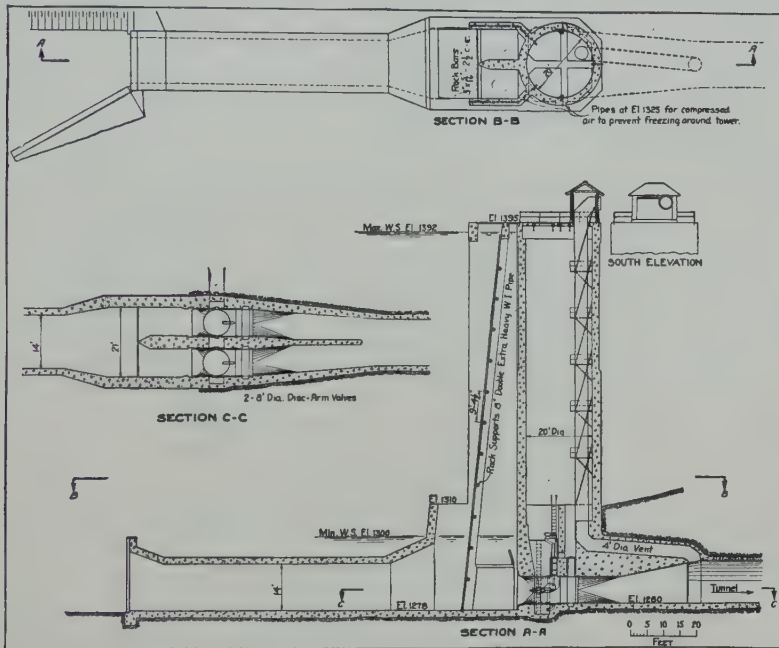


FIG. 220.—Reservoir outlet power and headgates—Davis Bridge plant, New England Power Company.

water the gates can be only partly opened in supplying the canal, and the excess reservoir head must be used up in friction and turbulent flow through the gates, obviously an unsatisfactory arrangement, if power is of value.

Where power is to be developed at or near the reservoir, it is, therefore, advantageous to use a pipe outlet placed below low-water level and thus utilize the full available head at all times. The control of flow will, therefore, be by sluice gates or some form of penstock gate. The racks must be of sufficient height to cover the range in draft of the reservoir, and the gate chamber becomes virtually a tower or gate well.

A good example of construction of a large reservoir outlet tower is shown in Fig. 220, at the Davis Bridge plant of the New England

Power Company. As will be noted, at this plant a range of water level of about 92 ft. is provided for, requiring racks 21 ft. wide by 110 ft. deep. As the bottom of the racks is about 80 ft. above the river valley and the reservoir is very large, little cleaning of racks is required. The inside of the main part of the tower, which is a dry circular well, is 20 ft. in diameter and 115 ft. deep. At the bottom of this well are two 8-ft. disk-arm-type Dow valves operated by motors mounted directly above and supported by the valve housing. These valves are

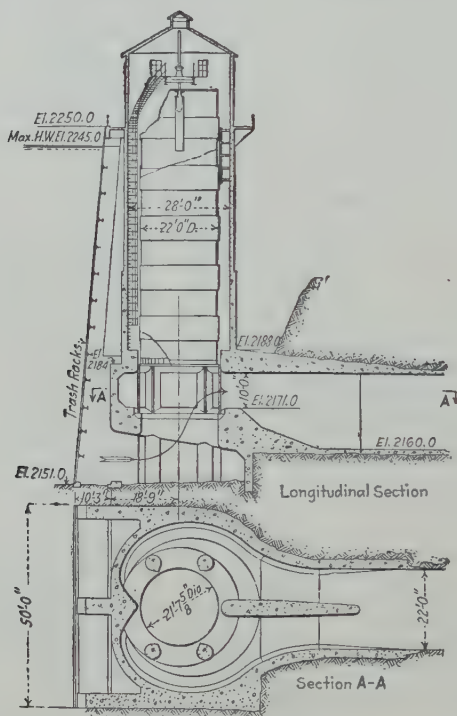


FIG. 221.—Cylinder gate—Big Creek 3 plant, Southern California Edison Company.

connected by a heavy cast-iron conduit with the upstream side of the tower and the tunnel on the downstream side. Provision is also made to bulkhead off both upstream and downstream ends of each valve for repair purposes. Another important feature is the 4-ft. steel vent pipe extending from the tunnel to the top of the tower, through the circular chamber.

The cylinder gate¹ may be adapted for use in the case of gate towers at reservoir outlets. Figure 221 shows such a gate at the intake of the 21-ft. tunnel of the Big Creek 3 development of the Southern California Edison Company, installed in 1924. This gate is 22 ft. in diameter

¹ Report of Hydraulic Power Committee, Nat. Electric Light Assoc., p. 71, 1924.

and 77 ft. high, operating inside a concrete gate tower 28 ft. in diameter and designed for a maximum discharge of 3000 sec.-ft. The gate shell is in tension to avoid interior bracing and make stresses more definite. Water passes upward through the gate orifice (the cylinder remaining filled) and, when the cylinder is raised, passes radially outward into a kind of scroll chamber surrounding the gate, and thence to the tunnel. It is intended only to be used either shut or fully open, the control of the water being by means of gates at the other end of the 6-mile tunnel.

A simpler form of cylinder gate is also used as an auxiliary spillway gate from the forebay of the Kern River plant of this same company, installed in 1920. This consists of a steel cylinder 8 ft. in diameter by 22 ft. high. The lower end of the cylinder is fitted with a bronze seat ring closing against a similar ring around the 8-ft.-diameter discharge hole in the floor of the forebay. When the gate is seated, water surrounds the cylinder to its full height, pressures are balanced, and the weight of the gate alone must be overcome in lifting. Operation is by electric motor, and no excessive vibration occurs at any gate opening.

Balanced or needle valves, such as the Johnson valve, described on page 483, are frequently used as gates for free outlet discharge under high heads. Their chief advantage is in eliminating chattering, usual with the ordinary sluice gate when partly open, and also in the comparative ease with which the gate may be operated under the high velocities incident to free discharge.

GATES IN DAMS

Gates in dams, aside from those used as headgates for the power development, are frequently required to aid in the passage of flood water. While a free and unobstructed spillway is generally the best and most dependable means of handling flood water, in some cases the range in height of water required by the spillway may not be available. Thus, at the outlet of a large reservoir, the use of a spillway control may be impracticable, because of the need to conserve the full depth of water in the reservoir. Flashboards might perhaps be used for the upper few feet of level, but, generally, to obtain adequate control, large gates in the dam would be required; in fact, most of the length of such a dam should be gate opening.

On large streams it is usually impracticable to handle large floods by gates alone, and the spillway must be the chief means to this end. On the other hand, a few gates may be of great service in handling ordinary freshets in such a way as to avoid carrying away the flashboards. Use may also be made of such gates to lower the water level at the dam temporarily, while flashboards are either being placed on or being removed from the dam. Such gates are best located in the dam, often in the spillway section, at an elevation low enough to obtain a good discharge head.

Crest gates, between piers, give an opportunity to utilize more discharge head than that afforded by the spillway. The piers, however, make an obstruction—unless very long gates are used—which may be dangerous where logs or ice frequently pass down the river. Such gates may be of timber or steel and concrete, as previously described, and the gate control by individual hoists or a traveling crane. Each of the steel crest gates at the Keokuk dam covers an opening 30 ft. wide by 11 ft. 3 in. high, between 6-ft. concrete piers, and is hoisted by a traveling

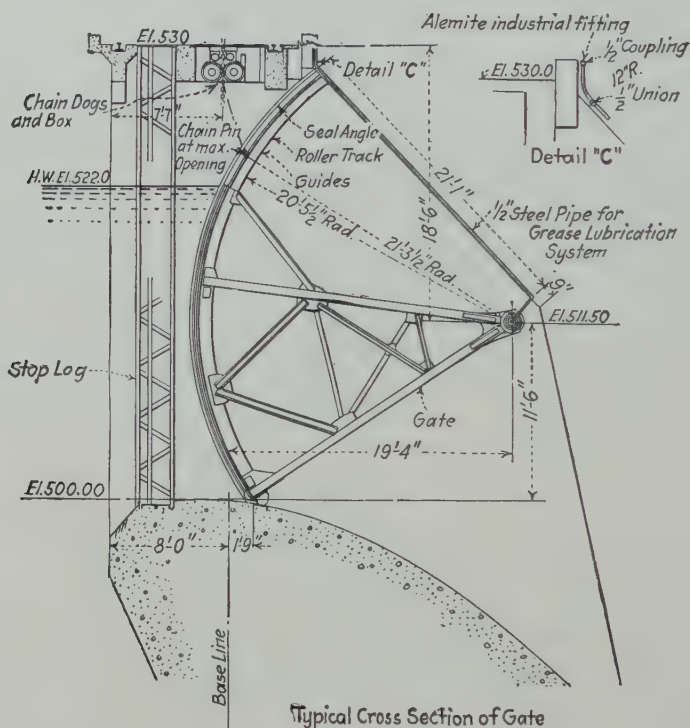


FIG. 222.—Tainter gates at Bartlett's Ferry plant.

gantry crane. To keep the gates from freezing, a compressed-air system, introducing air through pipes near the bottom of the gates, has been used successfully since 1918. Several heavy jams of ice have occurred here, notably one about 15 ft. deep, but in all cases the ice has broken up and passed through the gate channels without clogging them.¹

Tainter gates of the sector type, also called radial gates, are often used for crest gates. In this type of gate, the pressure of the water causes no resistance to opening or closing except the slight friction of the pivots and of the sealing strips at the sides. Hence, the weight of the

¹ Report of Hydraulic Power Committee, Nat. Electric Light Assoc., p. 18, 1924.

gate only has to be controlled by the operating mechanism. In closing the gate, its weight is ordinarily sufficient to overcome the slight friction and no mechanical effort is required.

In Fig. 222 is a cross section of one of the Tainter gates at the Bartlett's Ferry, Georgia, plant, constructed during 1925 (see also Figs. 301 and 302, Chap. XIII). There are 19 of these gates, each 25 ft. long, controlling a height of water of 22 ft. and operated by a gantry crane. The gate sills are of wood, and the side seals are steel angles with rubber belting. An interesting detail of these gates is the pipe system for grease lubrication of bearings.

Where wooden sills are likely to be bruised by ice or logs shooting under the gate, an iron or steel sill should be used. A desirable type of bottom seal is a double layer of rubber belting, three to five ply and 4 or 5 in. wide, parallel to the gate face and extending somewhat beyond it. These layers fold under the weight of the gate and usually press against a channel-iron seat, which is also flush with the bottom surface of the opening.

Tests to determine the coefficient of friction of rubber belting on structural steel and concrete were made by the Pacific Gas and Electric Company.¹ Tests were also made to determine the lifting forces required for radial gates, finding these to be from 88 to 100 per cent of the total weight of the gates.

Tainter gates are built in large sizes, a few of the larger installations being as follows:

Development or owner	Size, feet	
	Width	Height
Cheoah-Tallassee Power Company.....	25	19
Bartlett's Ferry, Georgia.....	25	22
Mitchell dam.....	30	15
Pacific Gas and Electric Company.....	20	11.5
Pacific Gas and Electric Company.....	28	22
San Joaquin Light and Power Company.....	20	14

The weight of the larger steel gates is from 50 to 60 lb. per square foot of gate area.

For greater depths an ogee sill is required, so that the water sheet will drop down and permit a suitable gate radius. With a short radius the load on the gate piers will slope decidedly upward and require special provision of rods and anchors and piers of large volume.

¹ *Report of Hydraulic Power Committee, Nat. Electric Light Assoc., pp. 53-55, 1924.*

The use of Tainter gates for submerged openings is generally undesirable, owing to the difficulty of making the gate water-tight on all four edges. Unless a water-tight sliding gate is installed above the radial gate, considerable waste of water will occur.

There is possible danger in the use of Tainter gates upon dam crests where logs or ice in large quantities may suddenly come down the river and jam in the gates so as to prevent raising them. This occurred in 1911 at the Rothschild dam on the Wisconsin River,¹ resulting in a serious washout at one end of the dam and heavy resultant expense. There is also some danger of Tainter gates being frozen into the ice and rendered

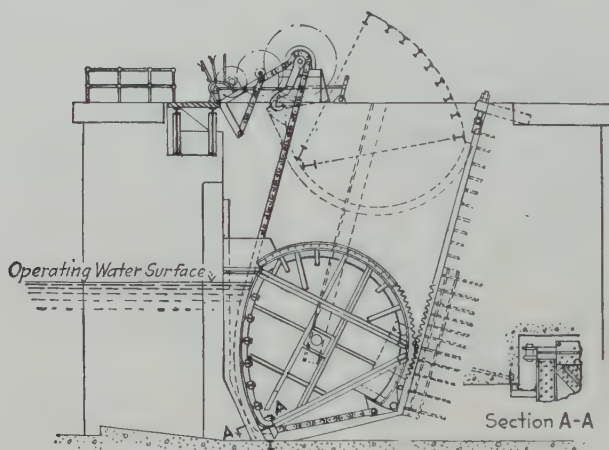


FIG. 223.—Rolling sector gate—Spokane Upper Falls station, Washington Water Power Company.

immovable in cold climates unless the ice is kept constantly chopped away.

The **sector roller gate**,² developed by Greisser of the Washington Water Power Company, Spokane, is installed for use with heights of water from 10 to 25 ft. and lengths of 50 to 100 ft., a range just beyond the field of the Tainter gate and overlapping that of the Stoney gate, which is suitable for greater heads but more limited spans. Its arrangement is shown diagrammatically in Fig. 223. Its load is applied in a direction with a downward slope and, hence, is transferred to the base of the dam more directly than that of the Tainter gate. It differs from the German roller dams which use a cylinder as the load-carrying member, while the Greisser type utilizes trusses for this purpose.

Both Tainter gates and roller gates are readily accessible when open and discharge a relatively large amount of water with small gate movements.

¹ MEAD'S "Water Power Engineering," p. 652.

² Report of Hydraulic Power Committee, Nat. Electric Light Assoc., p. 68, 1924.

Cylinder Roller Gates.—An installation of two German "M. A. N."-type roller gates manufactured by S. Morgan Smith Company was made at the Bellows Falls plant of the New England Power Association, completed in 1928.¹ These gates are made up of steel cylinders, each $13\frac{1}{2}$ ft. in diameter and 121 ft. long over all. To the cylinders are attached steel aprons provided with oak sealing strips, which when the gates are closed bear upon steel sills, making a height of gate of 18 ft. Tightness at the ends is obtained by oak seals bearing upon hollow-plate steel faces set in the concrete piers.

A motor-driven hoist of 192 tons capacity rolls the gate up an inclined rack by a chain connection at one end, at a rate of about $\frac{1}{2}$ ft. per minute. With the gate open a clear opening of 115 ft. long by 25 ft. high is available.

This type of gate, while relatively new in the United States, has been used in Europe for some time under a wide range of conditions, including the severe winter cold of Norway. It is easily kept free of ice by a moderate amount of heat properly applied. At Bellows Falls, freezing at the end seals is prevented by space heaters (about 8 kw. for each gate) placed in the cellular steel facings of the piers. No special tendency of the gates to freeze along the bottom seal has occurred, but the gates are opened a few inches once each week to insure readiness to operate them when required.

Submerged sluice gates in dams are often used, as previously noted, where flashboards are carried, for their protection from ordinary freshets. In Fig. 224 is shown the arrangement of such gates at the Vernon dam of the New England Power Company on the Connecticut River. These gates replaced original cast-iron gates which were too light and warped and cracked, one gate completely failing. The new gates installed in 1916-1917 are of semi-steel and of box design, operated by oil cylinders, and designed for stresses, both tension and compression, of not over 800 lb. per square inch. There are eight of these gates, each 8 ft. 10 in. high by 6 ft. 10 in. wide, in clear openings, located under the spillway section of the dam and reached by means of a passage through the dam from the power house.

Flashboards 8 ft. in height are carried at this dam and these sluice gates are operated probably a dozen times a year. The operating pressure required in opening indicates a friction coefficient of about 0.7.

The head on these gates averages about 35 ft., and they will discharge about 20,000 sec.-ft. or about 3.2 sec.-ft. per square mile of drainage area (6300 sq. miles). This is approximately the flow equaled or exceeded 15 per cent of the time.

At Turners Falls (7100 sq. miles) on the same river 7-ft. flashboards are carried; 12 sluice gates are located in the spillway section of the dam,

¹ *Jour. Boston Soc. Civil Eng.*, March, 1930, pp. 74-75.

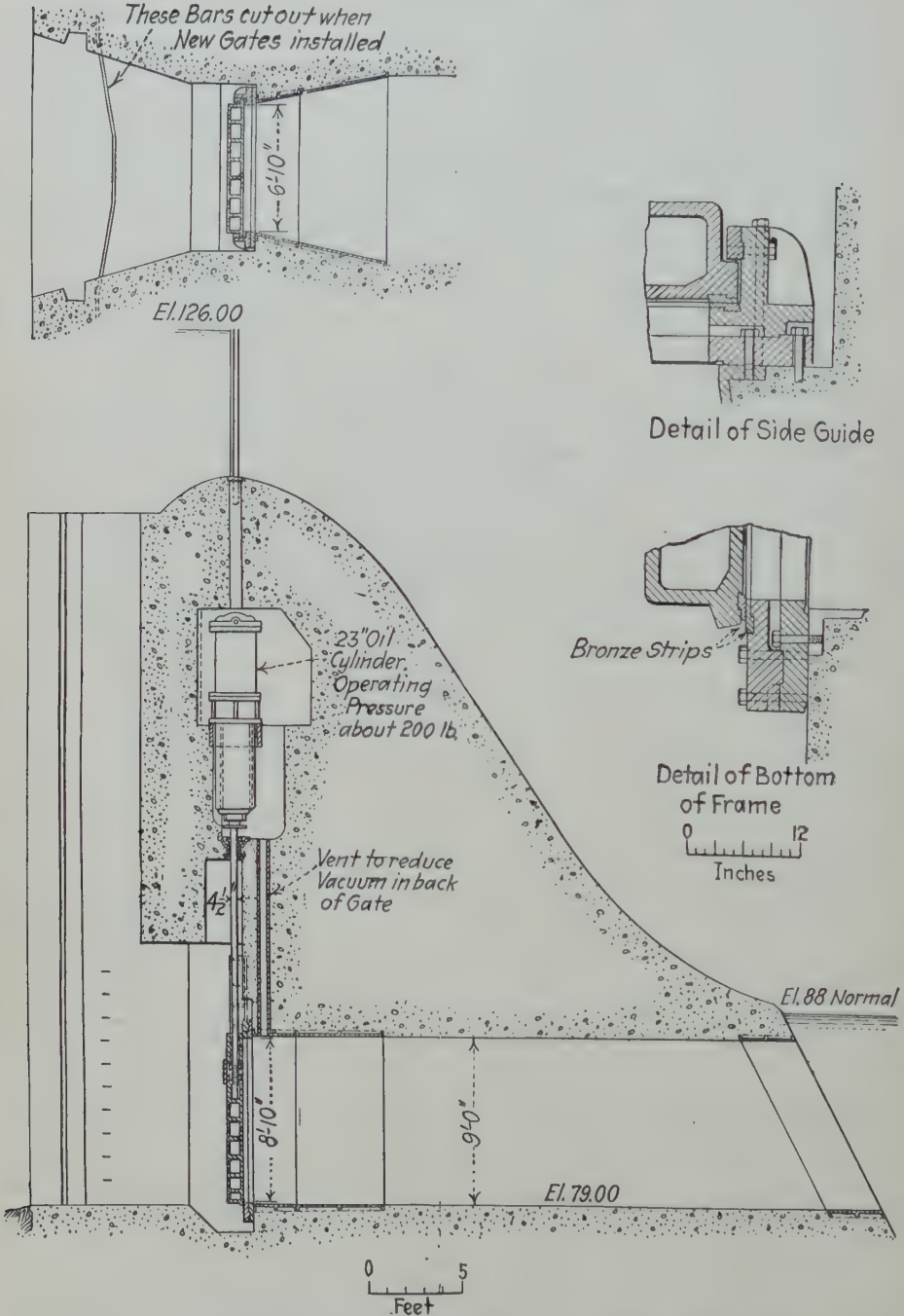


FIG. 224.—Sluice gates in dam—Vernon plant, New England Power Company.

six 8 ft. by 7 ft. 9 in. and six 7 ft. by 8 ft., operated mechanically, with a discharge capacity of about 24,000 sec.-ft. or about the same flow per square mile as at Vernon.

FLASHBOARDS

The most common device to provide a spillway crest which will automatically be lowered in times of flood is the use of flashboards. In their simplest form flashboards consist of one or more tiers of boards, usually 1-in. rough stock, held in place by iron pins which are supported in vertical sockets in the crest of the spillway. The boards are usually loosely fastened to the pins by staples or wires so that they will not float away or drop down when for any reason the water level back of them is lowered. Flashboards thus provide pondage at those times, particularly of low water, when it is of great value, without requiring a decrease of head below the normal. The pins are commonly so proportioned that if water flows over the boards to a considerable amount, usually predetermined, the pins will bend over, let the boards pass downstream, and permit a free discharge over the crest of the spillway. The pins may later be taken from the sockets, straightened, and used again.

Sockets for the pins are usually sections of pipe, with internal diameter slightly larger than the diameter of pin, set vertically in the concrete of the spillway.

Instead of solid pins, iron or steel pipe may be used, particularly for high flashboards, thus requiring much less material. After bending over, however, pipe is difficult to bend back into shape and pins are preferable for boards up to a height of about 3 ft. In proportioning these pins, their spacing will be fixed by the strength of the boards, using a relatively high allowable bending stress under the assumed maximum conditions of, say, 1800 lb. per square inch, and the size of pin by the pin spacing and the extent of surcharge under which the pins are to bend over, with a stress somewhat beyond the elastic limit. The loading which will cause bending over should preferably be checked experimentally.

Flashboards can be kept fairly tight and leakage avoided by sprinkling ashes back of the boards occasionally. While the boards will usually be lost once or more during the season and the process of replacing involves some labor, the use of flashboards usually constitutes an economical means of pondage control. With high boards, as previously noted, sluice gates in the dam will be very helpful to avoid losing the boards under an ordinary freshet pitch. In the process of replacing the boards during a falling-river stage, sluice gates in the dam are again useful to draw the pond level down temporarily, so that the boards may be replaced weeks before this would be possible without them.

High flashboards will require additional support other than that from pins alone. Figure 225 shows the arrangement of the 6-ft. boards at Plant 4 of the New England Power Company on Deerfield River, Vermont. The pipes are 3-in. standard wrought iron spaced 2 ft. on centers. Groups of three adjacent pipes are supported by short horizontal steel angles fastened to the pipe near the level of the top of the boards and held in place by diagonal 2-in. pipe braces with a swivel joint at its lower end. When it is desired to remove the flashboards, each section of three pins is lifted out by means of the cableway shown in Fig. 225. The pipe braces fall down in place and are again used when the boards are replaced. In this case an angle in the dam complicates the arrangement of the



FIG. 225.—Six-foot flashboards—Plant 4, New England Power Company.

cableway control. A similar scheme is used at the Vernon dam of the same company, where 8-ft. flashboards are carried.

At the Turners Falls dam on the same river 7-ft. flashboards are carried, shown in detail in Fig. 226. In this case the boards are in 6-ft. sections divided horizontally into a 4-ft. and 3-ft. panel, with 4-in. pipe spaced 3 ft. on centers and supported 5 ft. up by a horizontal 4- by 4-in. ranger, which is itself supported every 6 ft. by a diagonal strut of the same size.

Another scheme sometimes used for tripping inclined props or flashboard supports is to have them connected with a cable running to shore, which can be pulled sufficiently to disengage the props from their supports, thus allowing the pins to bend over and the boards to go out.

A more elaborate arrangement of high flashboards so constructed that the individual boards may be removed for ordinary regulation, while the supporting posts and props can be tripped and boards go out in

freshets, is in use at the Kern Canyon dam of the San Joaquin Light and Power Company.¹ The posts, boards, etc., are held from washing away by chains and are used again.

Flashboards for Reservoirs.—With a large reservoir the storage capacity in the upper level eliminates the possibility of a dangerously sudden rise of water level. Furthermore, owing to the value of the stored water, it is not desirable to draw the water level down temporarily to replace flashboards. Hence, a type of board is required which can be replaced with a considerable depth of flow over the crest and yet be

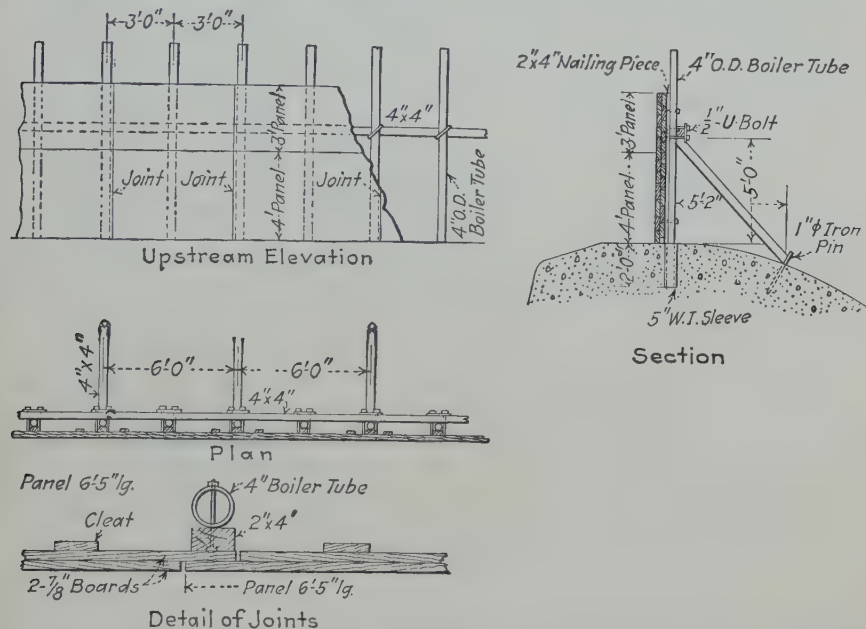


FIG. 226.—Flashboard details—Turners Falls dam, Turners Falls Power and Electric Company.

removed with certainty, ease, and quickness. The objection to piers on the spillway crest, due to possible clogging with logs, ice, etc., is greatly reduced with a reservoir. Hence, a good arrangement is that of stop logs supported by needle beams which can be tripped from a bridge between the piers, as used at the Somerset and Davis Bridge reservoirs of the New England Power Company.² The arrangement at the latter reservoir is shown in Fig. 227. This has a circular spillway 160 ft. in diameter (see Fig. 289, Chap. XIII) discharging through an outlet tunnel. There are 16 radial piers, and the span between each set of piers, of about 26 ft., is broken up by three needle beams ($5\frac{1}{2}$ -in. Carnegie

¹ Report of Hydraulic Power Committee, Nat. Electric Light Assoc., p. 7, 1924.

² Dow, E. A., "Mechanical Features Affecting Operation of Hydroelectric Plants," *Jour. A.S.M.E.*, 1925, pp. 415-460.

cross-ties) supporting the stop logs. The beams can be tripped from the bridge by means of the latch at the top, thus releasing the stop logs. The former are still held by the chains, however, and can be pulled up and replaced at any time, the tops being returned to the latch and the bottom ends allowed to drop into the sockets at the bottom. Normal regulation at this reservoir can be obtained by pulling up a few boards.

Collapsible Flashboards.—These are usually of the hinged type, so arranged that different lengths between piers can by suitable mechanism be lowered to a flat position, leaving the spillway free. Such an arrangement is used at the Burton dam of the Georgia Railway and Power Company. The spillway consists of eight 22-ft. openings between 3-ft.

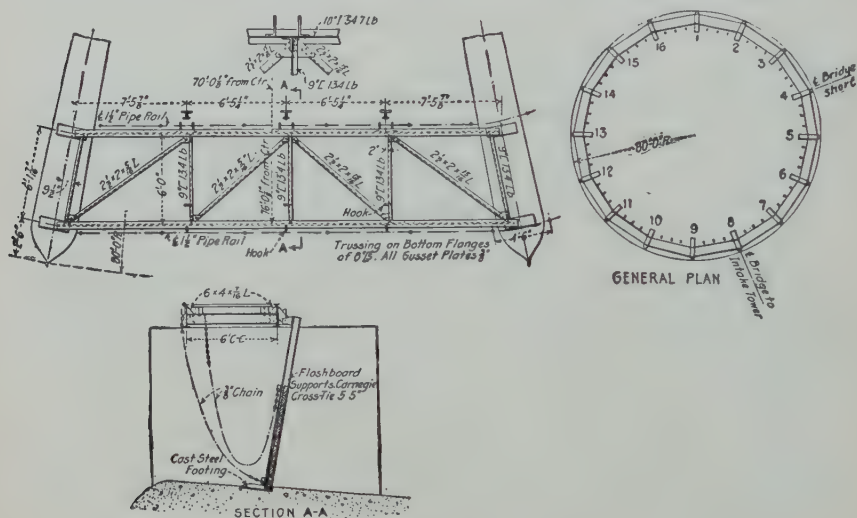


FIG. 227.—Flashboards on circular spillway—Davis Bridge development.

piers, with a walkway over the piers. The boards hold a 6-ft. head over the spillway and are controlled by water-operated winches and steel cable. A master float switch is designed so that if the pond level rises 1 in. above normal, the central two sets of boards are fully lowered. With another inch of rise in the pond, the second set of boards goes down, and so on until with a 4-in. rise in pond all eight boards are down. As the water recedes from its maximum level, the sections of boards are automatically raised in reverse order.

At the Mathis dam of the same company, similar boards are used, 16 sets, each 15 ft. long. Nine of these sets are controlled by motors from the station switchboard. The other seven are counterweighted by a heavy concrete roller or drum, which rolls up inclined tracks located under the ends of the roller and unwinds a cable supporting the upper end of the flashboard, thus lowering it, the action being controlled by the height of water in the pond and consequent pressure against the board.

It has been found difficult to control the spill exactly as desired with these boards, as operating conditions vary considerably from time to time due to rust accumulations, friction, etc., and the electric control scheme is considered the better.

Recently, collapsible flashboards built of steel in sections, hinged at the base so that they can be lowered to the crest of the spillway, have been used at the Bristol plant of the Public Service Company of New Hampshire.¹ (See Fig. 228.) The sections are built of $\frac{5}{16}$ -in. steel plate backed by 5-in. H-beams. Each section is 7.5 ft. high and about 5.5 ft. wide, with a 5-in. wrought-iron pipe (filled with concrete) welded along the bottom edge. Angles at the top form a lip which engages a supporting wooden strut resting upon the spillway crest. The hinge

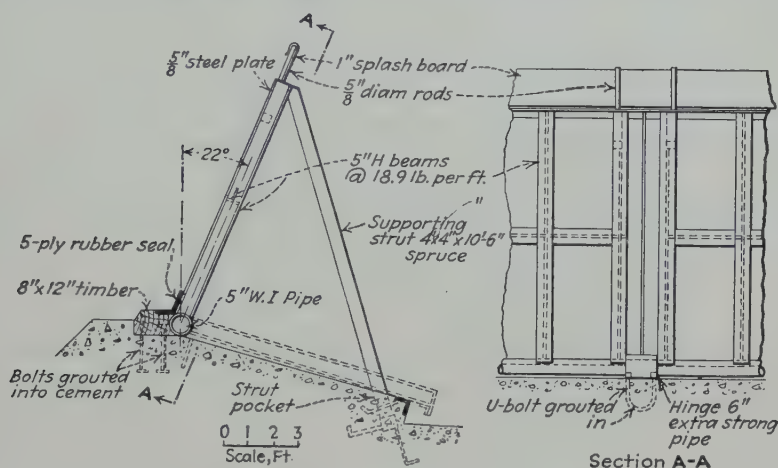


FIG. 228.—Hinged flashboards at Bristol, N. H., plant.

consists of a section of 6-in. pipe welded to a plate, anchored to the spillway crest. The joint between sections is made tight by a wedge-shaped wooden sealing strip. The opening between the pipe base and the dam is sealed with a rubber strip. An additional foot of boards can be placed at the top as shown in Fig. 228. Raising and lowering the boards are aided by two cableways. A total length of 263 ft. of spillway is installed with these flashboards, in three bays separated by concrete piers.

The Stickney automatic crest gate is arranged as shown diagrammatically in Fig. 229. It is built in sections between piers and has two leaves or faces at right angles to one another. The upper leaf holds the water level above the spillway at ordinary river stage by virtue of the greater pressure on the lower leaf, which pressure is applied through openings in the dam as shown. With proper relative length of leaves OA and OB and the use of a counterweight W as shown, the balance is such that at

¹ "Hinged Flashboards for Headwater Control," *Elec. World*, July 16, 1932, p. 79.

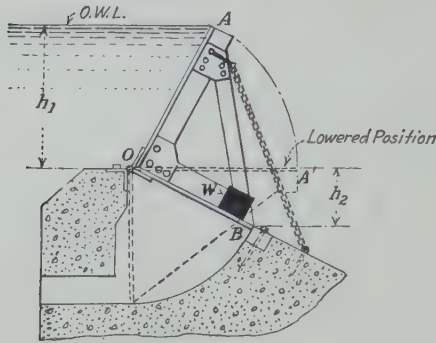


FIG. 229.—Stickney automatic crest gate.

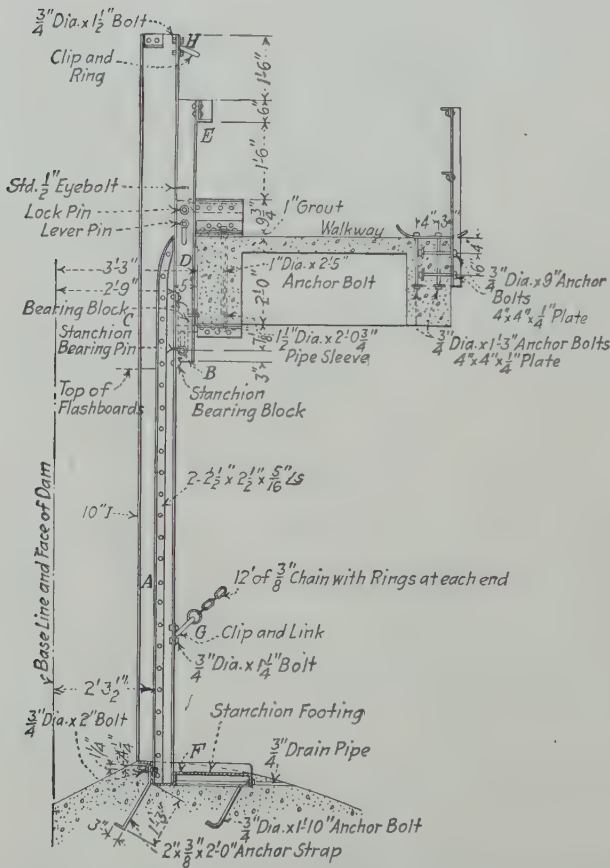


FIG. 230.—Stanchion type of flashboards—Jackman development, Contoocook River, New Hampshire.

high water the increased pressure on the upper leaf will cause the gate to move to the lowered position (shown dotted in Fig. 229). The ends of both leaves must be sealed, as well as the bottom of the lower leaf.

Such a type of gate is obviously limited in use to relatively low-head control.

The drum and bear trap drums are similar in principle to the above gates. The drum gate is the standard crest gate of the U. S. Reclamation Service.

Stanchion Type.—The deck or stanchion type of flashboards is a type requiring piers with bridge spans, the boards being held by vertical stanchions or posts which are pivoted near the top and so arranged that they may be swung to a horizontal position, thus clearing the channel between the piers and allowing the boards to go out. A recent installation of this kind at the Jackman development of the Public Service Company of New Hampshire on the Contoocook River (constructed in 1925), is shown in Fig. 230. The total span between dam abutments is 112 ft., divided by three 24-in. piers into four approximately equal spaces. Each space between piers has five stanchions, which are 4 ft. 5 in. on centers. The stanchions *A*, constructed of steel I's with angles attached to hold the boards (Fig. 230), rotate on a bearing pin *B*, which moves in a slot in the heavy block *C*, and a lifting post *D*. The latter is raised by a jack inserted at *E* and remains in a vertical position. When the stanchion clears the footing at *F*, it rotates about the pin *B*, the boards go out, and the stanchion is raised to the horizontal position by the chain attached at *G*. When the boards are to be replaced, the stanchion is lowered and brought to the vertical position by means of a pull by block and tackle on the link *H*, the stanchion footing at entrance (downstream) being flared so as readily to catch the foot of the stanchion, which finally drops into position in the footing recess. The usual height of flashboards carried will be 9 ft., with pond level 8 ft. above crest of dam, and they will be 2- and 3-in. boards, dressed on downstream side.

Stanchion-type flashboards 13 ft. high were also installed at the Bellows Falls plant of the New England Power Association in 1928,¹ consisting of 16-in. I-beams spaced 6 ft. 3 in. on centers, with 4-in. plank boards, in two bays of 121 ft. length and one bay of 100 ft.

SIPHON SPILLWAYS

The siphon spillway is a device for obtaining discharge without mechanism, and with small variation in head-water level, through openings near the top of the dam under a head approximately the height of the dam, keeping in mind the practical limitations in use of a suction head of more than approximately 25 ft. (theoretically 34 ft.).

¹ *Jour. Boston Soc. Civil Eng.*, March, 1930, pp. 75-76.

In Fig. 231 are shown details of the arrangement of one of the siphon spillways at the Bartlett's Ferry, Georgia, plant, installed for automatic regulation of reservoir level (see also Fig. 301, Chap. XIII). There are four of these siphons, with throat crests at elevations of 520.00, 520.08, 520.17, and 520.25, respectively, and each throat crest area 11 ft. 6 in. wide by 4 ft. 0 in. high. The center of the outlet area on the downstream face of the dam is at an elevation of 488.5. Each siphon has an air inlet in two sections, each 12 in. high by 4 ft. 6 in. long, covered by a steel grille, with top at the same elevation as the siphon-throat crest. These siphons are built with an upward curve or trap with its crest at an elevation of 495, thus sealing the upper half and requiring less removal of air before

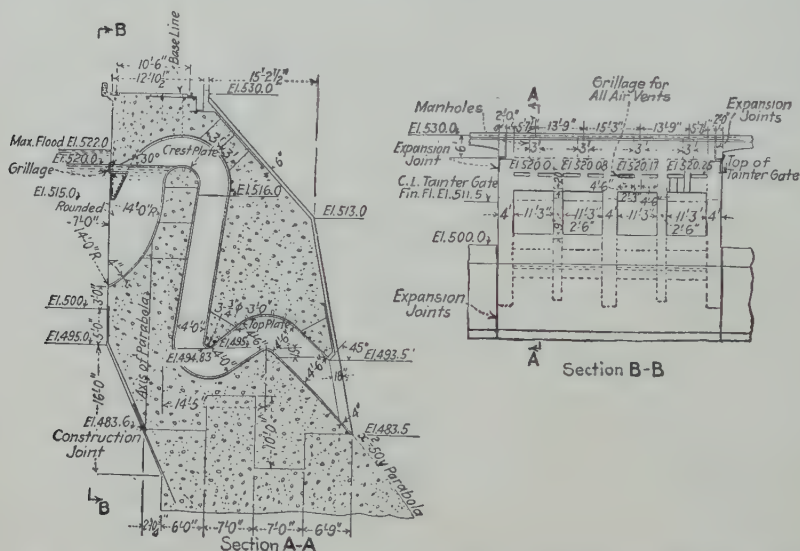


FIG. 231.—Siphon spillway—Bartlett's Ferry plant. (Courtesy of Stone and Webster Engineering Corporation.)

the siphon becomes fully operative. The range of pond level during the operation is from an elevation of 520 to 522 at maximum flood level. Normally, therefore, the operating head is about 32 ft., with a total siphon-throat crest area of 184 sq. ft.

Test of a model of one of these siphons on a scale of 1:5 made at the Hydraulic Laboratory of the Massachusetts Institute of Technology in June, 1925, showed a value of coefficient of discharge of about 0.54. This is somewhat lower than the usual value (0.6 to 0.65), due probably to the resistance to flow offered by the additional curve in the lower part of the siphon. The experiments indicated, however, that the siphon will prime rapidly and give a close regulation of water level at the dam, as desired. Using this coefficient the total discharging capacity of the four siphons under a head of 32 ft. would be about 4500 sec.-ft., or about

1.07 sec.-ft. per square mile for the 4200 sq. miles of drainage area above the Bartlett's Ferry plant.

Siphon spillways have been used in a number of developments and furnish an effective means of increasing discharge above that of a weir or ordinary spillway. Provision must be made, however, to prevent the entrance of ice or debris which might clog the siphon. In cold climates there is danger of the siphon becoming inoperative on account of freezing, unless some means is taken to prevent this.¹

LOGWAYS

In many parts of the country where logging is still carried on, provision must be made for passing logs by the dam. Where plenty of water

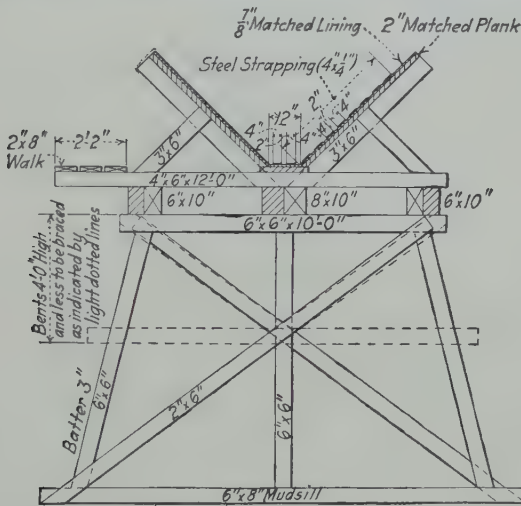


FIG. 232.—Log sluice—Millinocket, Me. Typical bent.

is available in low-head developments, the logway may consist simply of a large, relatively shallow timber gate, usually at one end of the dam, in the abutment section, discharging into a flume of approximately the same width which leads to the foot of the dam. Such an arrangement requires a large amount of water, however, as the gate must be wide open when in use. By arranging the gate so that it lowers in opening, less water will be required, particularly where the pond level at the dam fluctuates materially.

To avoid waste of water, particularly at high-head developments, specially constructed logways or log sluices are often used as shown in section in Fig. 232. This is installed at the development of the Great Northern Paper Company, Millinocket, Me., where a head of 110 ft. is

¹ See also *Report of Hydraulic Power Committee*, Nat. Electric Light Assoc., pp. 13-18, 1924.

utilized. The log sluice is about 2800 ft. long, with a total drop of approximately 112 ft. It is controlled by a gate (rising to open) 8 ft. wide by $6\frac{1}{2}$ ft. high covering an opening $6\frac{1}{2}$ by $5\frac{1}{2}$ ft. (decreasing to a depth of 5 ft. at the wooden sluice), with about 200 ft. of rectangular concrete section, transforming into the section shown in Fig. 232. The slope is as follows:

Station	Slope	Remarks
0 to 2 + 80	0.004	Concrete section
2 + 80 to 4 + 60		Vertical curve
4 + 60 to 13 + 45	0.050	
13 + 45 to 16 + 60		Vertical curve
16 + 60 to 27 + 70	0.035	

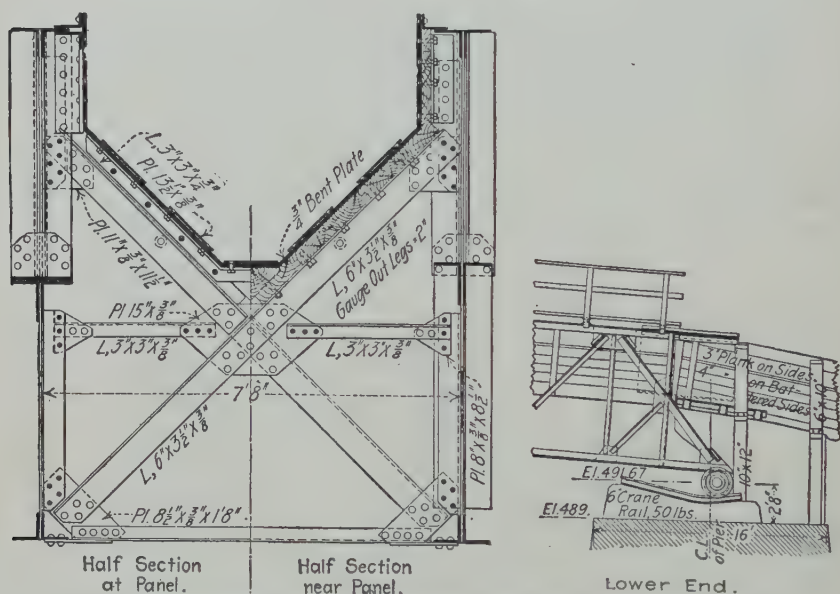


FIG. 233.—Adjustable log sluice—Azischohos dam, Magalloway River, Maine.

When in use, it requires about 200 sec.-ft. of water, and its capacity is about 125,000 ft. b.m. per hour when sluicing long logs of fair size.

Adjustable logways may be required in reservoir dams where considerable fluctuation of water level is required in utilizing storage. An interesting example of this type¹ is shown in Fig. 233 as used at the Azis-

¹ MOULTON, S. A., "Problems Encountered in Constructing the Azischohos Dam," *Proc. Nat. Assoc. Cement Users*, vol. 3, 1911.

cohos dam on Magalloway River, Maine, constructed and in use since 1911. This is located in a section between two of the buttresses of a hollow concrete dam, the opening being 7 ft. wide. A movable steel truss, span of 93 ft. 9 in., supports a V-shaped flume which discharges into a fixed V-section about 1 mile in length carrying the logs beyond a

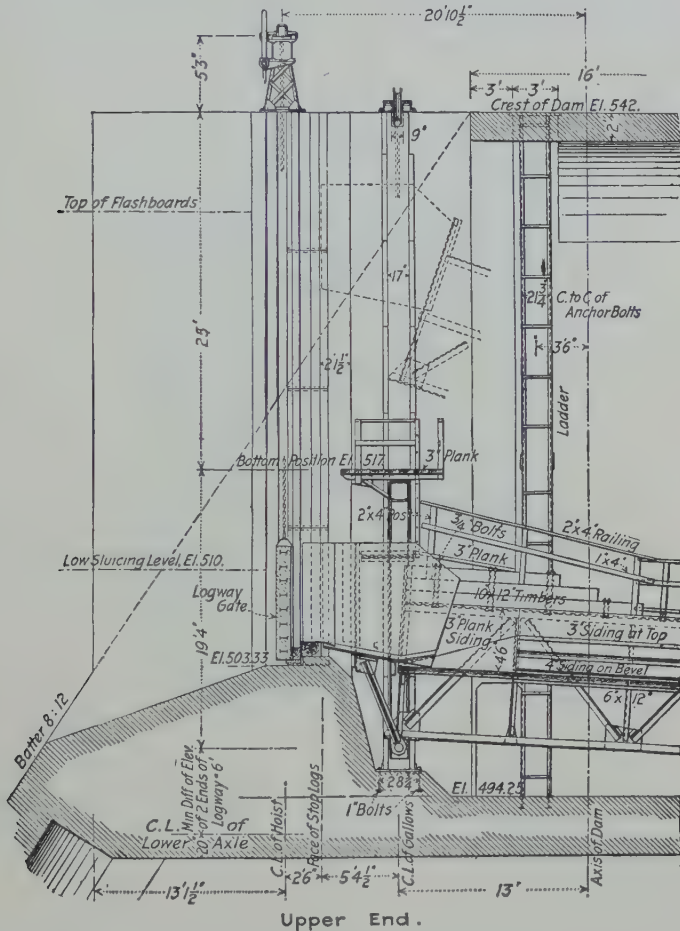


FIG. 233.—(Continued.)

series of rapids. The raising or lowering of the truss, which covers a range of 25 ft. in water level, is done by a 15-ton power crane, with a double-speed hoist for quickly handling the stop logs and the short section of spout set in the stop-log grooves, and overlapping the entrance mouthpiece to prevent leakage. The logway opening is closed with 12-by 16-in. hard-pine stop logs in cast-iron grooves, which also act as guides

for a steel-and-concrete drop gate which is used for controlling flow through the sluice.

Other details of the adjustable logway and of the steel flume will be noted in Fig. 233. Estimates indicate that, depending upon the sluiceway level, from 1 to 3 mill. ft. b.m. of logs can be handled in 10 hr.

The saving in use of water by this logway is very striking, the needs for log driving being reduced from about 2700 mill. cu. ft. formerly required during a season, to about 250 mill. cu. ft.

FISHWAYS

In many of the states, fishways are required by law in dams constructed upon natural waterways, so arranged that fish may pass freely up the stream. The essentials for a fishway are: (1) easy passage for fish, with uniform flow of water, a gradual ascent, and absence of high barriers; (2) a minimum use of water; (3) an entrance into which fish are readily directed; (4) durable and solid construction which will not be injured by freshets or which may be readily removed when not in use.

Types of Fishways.¹—There are three kinds of fishways in common use:

1. *Natural stream fishways*, where a small side stream of gradual slope is used, supplemented usually with a few concrete barriers, connecting the pond above the dam with the river below. It is easily surmounted by fish but requires an excessive amount of water, is often impossible to install owing to unfavorable local conditions, and has its entrance some distance downstream from the dam so that fish, in following the greatest flow of water, are likely to miss it.

2. *Pool fishways*, consisting of a series of pools with a drop of about 1 ft. between successive pool levels. It may be constructed of wood or concrete and arranged so that most of the flow may be through orifices between the pools, through which orifices the velocity of flow should be not over 6 or 8 ft. per second. The water in the compartments will be relatively quiet, thus affording a resting place for the ascending fish. With small narrow fishways, the discharge may be over the weir crest of each compartment.

3. *Inclined-plane fishways*, arranged like a long narrow box inclined on a grade of about 1 in 10, with various arrangements to check the flow of water and afford resting places for the fish. Wood or concrete construction may be used. A primitive, but effective type, is made by placing 1-ft. cross boards either perpendicularly or at an angle at 3-ft. intervals, and alternately at each side, so that water flowing down the incline on striking the projections rises in wave-like crests.

¹ Report upon the Alewife Fisheries of Massachusetts, Division of Fisheries and Game, Dept. of Conservation, pp. 59-63, 1921.

The installation of a fishway requires careful study and planning. If the pond level varies somewhat, the upper section of the fishway must be made adjustable, with an arrangement of either gates or stop boards to allow for different water levels.

The entrance to the lower end of the spillway should be at the foot of the dam, to accord with the locus of the greatest flow of water. This may require a return angle (in plan) of the fishway. Damage by floods may be minimized by a protected location, by suitable concrete construction, or by having a part of the fishway removable when not in use.

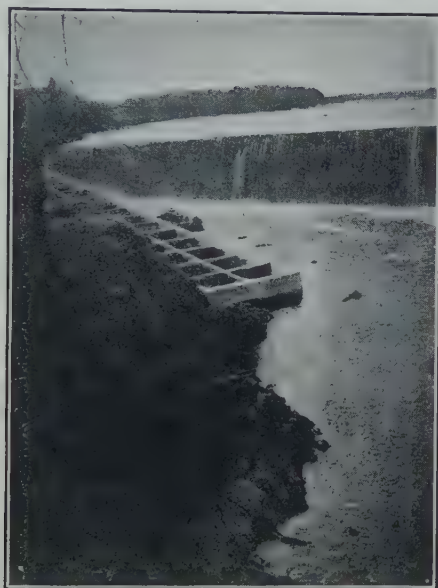


FIG. 234.—Fishway at dam of Essex County, Lawrence, Mass.

A simple form of straight-run, pool-type fishway is shown in Fig. 234, as installed at the dam of the Essex Company at Lawrence, Mass., on the Merrimack River. This has a general slope of 1 on 8 and is built of reinforced concrete in 8-ft. square bays with a water depth of about $2\frac{1}{2}$ ft. in each bay below a 4-ft. length of crest or weir, each crest being $\frac{1}{2}$ ft. lower than the preceding one and the water traveling laterally between two opposite bays, then downstream to the next pair of bays, and across this pair, etc. At the dam an adjustable timber chute is used to allow for variable water level.

Cost of Plant Accessories. *Gates.*—For gates of different types, including controls, the following unit costs, upon approximately a 200 per cent cost index, will be found useful for preliminary estimates of ordinary sizes. The cost of installation (usually 10 to 20 per cent, depending upon the size and type of gate) is not included.

Kind	Cost per Square Foot of Water Area
Wooden headgates and hoists.....	\$ 15
Tainter gates and hoists ^a	\$ 15 (of which about \$6 is for hoist)
Broome gates and hoists.....	\$ 40- 60
Metal sluice gates and hoists.....	\$ 75-100
Wicket gates and controls.....	\$ 75-100
Dow pivot gates and controls ^b	\$150-300
Cylinder rolling gates.....	\$ 80-100

Flashboards (including setting):

Pin and board type.....	\$2.00-3.00 per square foot
Stanchion type.....	\$4.00-5.00 per square foot
Racks (including setting).....	\$2.00-3.00 per square foot

^a The equation, $\text{Cost} = \$15 + \frac{450 - A}{40}$, where A is area in square feet, applies approximately.

^b Total cost of gates and controls is approximately $\$7000 + \$1700(D - 5)$, where D is diameter in feet.

CHAPTER X

SPEED AND PRESSURE REGULATION

GOVERNORS

General Features.—Speed regulation of hydraulic turbines is accomplished by altering the gate or nozzle openings of the turbine and consequently the amount of water supplied to it in proportion to the load, by means of an automatic speed-responsive governor.

Absolutely constant speed is not possible, since the automatic governor can function only with a change in speed.

The essential elements of an automatic governor for hydraulic turbines are (1) a speed-responsive device, usually fly balls, (2) compensating device, (3) relay apparatus, (4) valve mechanism, (5) regulating cylinder, (6) pressure pump, (7) pressure and receiving tank.¹

The governing force required for hydraulic turbines is large as compared with other prime movers, on account of the weight and nearly incompressible nature of the water and the inertia of the heavy gate mechanism required. It is impracticable to develop sufficient force in the speed-responsive device to operate the turbine gates directly, as the work to be done is about 3000 ft.-lb. for small turbines and 100,000 ft.-lb. and more for large turbines such as are now installed. Consequently the small force developed by the fly balls is relayed to a valve mechanism which controls the passage of a pressure medium, such as water or oil, into one or more regulating cylinders, which apply the relayed force of the speed-responsive element to the turbine gates.

The pressure medium used generally is oil, and the pressure applied is from 150 to 200 lb. per square inch. In order to keep the capacity of the necessary pumping equipment within reasonable limits, the oil or other pressure medium used is pumped into a tank containing an air cushion. This provides a reserve which can be drawn upon at a rate much in excess of the pump capacity and without reducing the pressure below the required amount if the equipment is properly proportioned. A receiving tank is required to hold the oil returned from the discharging side of the regulating cylinder until it is pumped back into the pressure tank.²

¹ Mechanical governors are so seldom used at present that a description of such governors seems unnecessary.

² Only the open-governor oil-pressure system will be described, as the vacuum system once used is now obsolete. The relative advantages of the two systems are fully described in the *Reports* of the Nat. Electric Light Assoc., etc.

On account of the momentum of the turbine-gate mechanism and the time required to accelerate or decelerate the water flowing to the turbine, the tendency of a governor is to move the gates farther than necessary to return the speed to normal after a load and consequent speed change. A successful governor must compensate for this tendency, otherwise hunting or racing will result. A compensating device consists of a dash-pot, often combined with springs interposed in the relay mechanism and operated from the regulating cylinder. If the regulating cylinder begins to move in one direction, it transmits a movement to the compensating

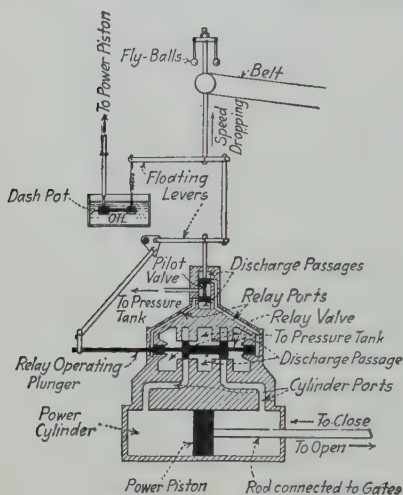


FIG. 235.—Automatic governor for hydraulic turbines. Diagrammatic arrangement.

device which has a tendency to reverse the action of the valve mechanism, thus preventing excessive overtravel. If these parts are properly designed and proportioned, the governor will move the turbine gates only to or slightly beyond the position which will result in normal speed for a changed load, and it is then known as a "dead-beat" governor.

In order to increase the sensitivity of the governor and reduce the force which the fly balls must develop, a pilot valve is introduced between the fly balls and the oil-regulating valve. The main regulating valve is designed to be hydraulically balanced in its closed

position when the pilot valve which controls the flow of oil to the main valve assumes its normal-speed position. The slightest movement of the pilot valve will disturb the hydraulic balance of the main valve and will cause it to admit oil to one or the other end of the regulating cylinder. Since the main valve is operated by the high oil pressure existing in the pressure tank, it will move quickly and positively without any outside mechanical force.

High-capacity governors require regulating valves of large dimensions in order to be able to pass the amount of oil which is often required in a very short time interval.

Figure 235 is a diagrammatic representation of the various governor functions described above, which may be explained as follows:

Assuming load on and falling speed, the belt-operated fly balls raise the floating-lever system and pilot valve so that pressure from the tank *via* the right-hand relay port moves the relay valve to the left, uncovering the left-hand cylinder port through which pressure from the tank

operates, moving the power piston to the right and opening the wheel gate. The compensating device consisting of the two dashpots and spring permits the initial movement of the floating-lever system but tends

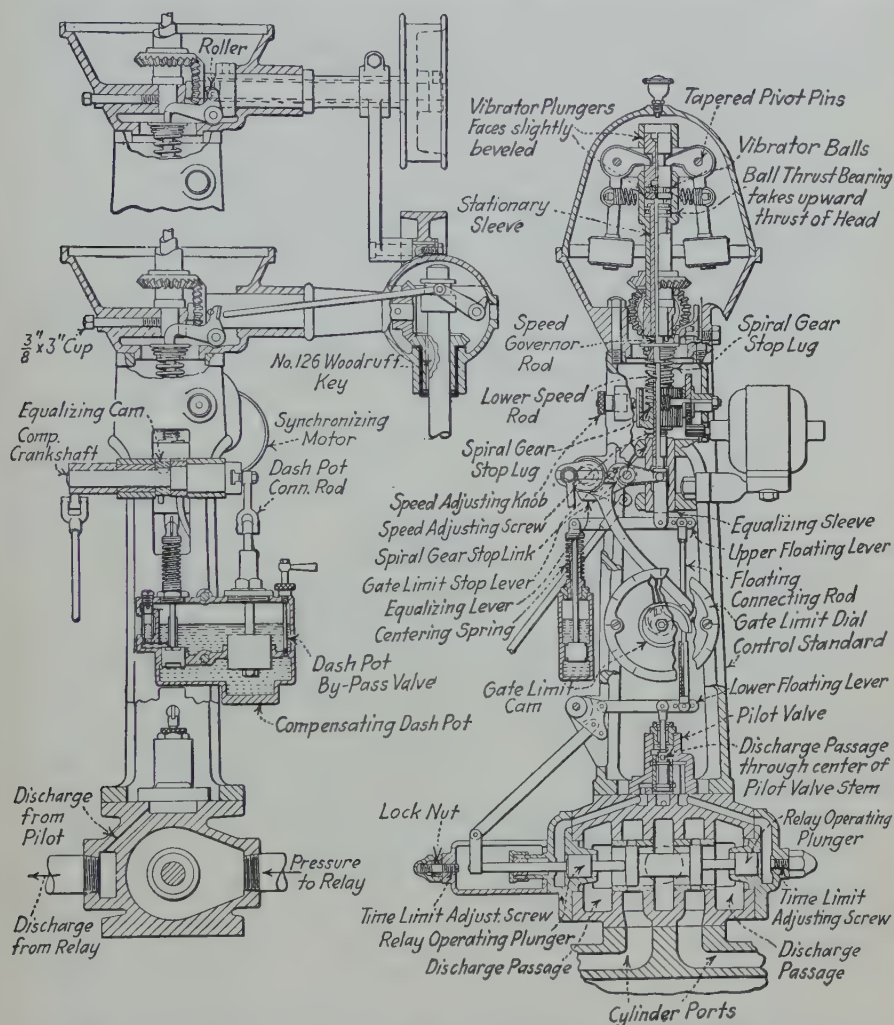


FIG. 236.—Control column and relay valve for oil-pressure governors—sectional detail—Woodward Governor Company.

at once to curtail this movement and thus prevents overtravel of the wheel gate.

With load off and rising speed, a similar cycle of events occurs, resulting in movement of the power piston to the left and closing of the wheel gate.

Figure 236 shows in detail the control columns and relay valve for the Woodward governor, for comparison with Fig. 235.

Auxiliary Devices.—In addition to the essential governor elements described above, most commercial governors are now equipped with some or all of the following auxiliary devices:

1. A device incorporated in the fly balls by means of which the speed of the turbine may be varied some amount above and below normal. If equipped with a small motor, which may be energized from the switchboard of a hydroelectric plant, this device is known as a synchronizing device.

2. A load-limiting device. This consists of an adjustable stop acting on the governor relay in such a way that the turbine gates will not open beyond a predetermined amount without lowering the speed of the unit.

3. A safety trip which is operated by the fly-ball collar when the speed rises above a predetermined amount. When the trip acts, the pilot valve is forcibly lifted and held in that position until the trip is manually released. This causes the main regulating valve to assume its "turbine-gates-closed" position.

4. An emergency-stop device consisting of a solenoid mounted on the governor stand which can be energized by closing a switch on the switchboard. The solenoid when energized immediately causes the lifting of the pilot valve and holds it in the lifted position, thus bringing the turbine to a definite stop in the shortest possible time.

5. Hand-operating equipment, consisting of some mechanical or hydraulic means of opening or closing the turbine gates when no oil pressure is available in the pressure tank.

6. A stop for definitely holding the turbine gates in the closed position. This may preferably be a part of the turbine equipment instead of the governor.

Automatic and Remote Control.—Some of the above devices, especially 1, 2, and 4, are often utilized in connection with automatic or remote-control hydroelectric plants which are frequently installed at the present time.

Some semiautomatic plants are started by a push-button control from a remote point utilizing the synchronizing and load-limiting devices for this, slowly bringing the unit up to speed from a fully closed position and then throwing it on the line. After the unit is in parallel, it may be controlled from the head-water level by means of floats, the gate opening being adjusted automatically to suit the flow in the river.

Full automatic plants are usually started and stopped by a predetermined elevation of the forebay water level by means of the solenoid described above, or sometimes by means of the synchronizing device. Such a full automatic plant should have a small capacity compared with that of the whole power system to which it supplies its power; otherwise frequency and voltage fluctuations may result which are unsatisfactory.

Governor Improvements.—No important improvements have been made on hydraulic turbine governors in recent years, except for the additions of the auxiliary and automatic devices mentioned. The workmanship on the governors, however, has been much improved, and this has resulted in the elimination of many difficulties formerly experienced.

Two new methods of fly-ball drive have been developed. In one of these the fly balls are mounted directly on the main turbine shaft, and their movement is transmitted to the relay by a small rod. The second

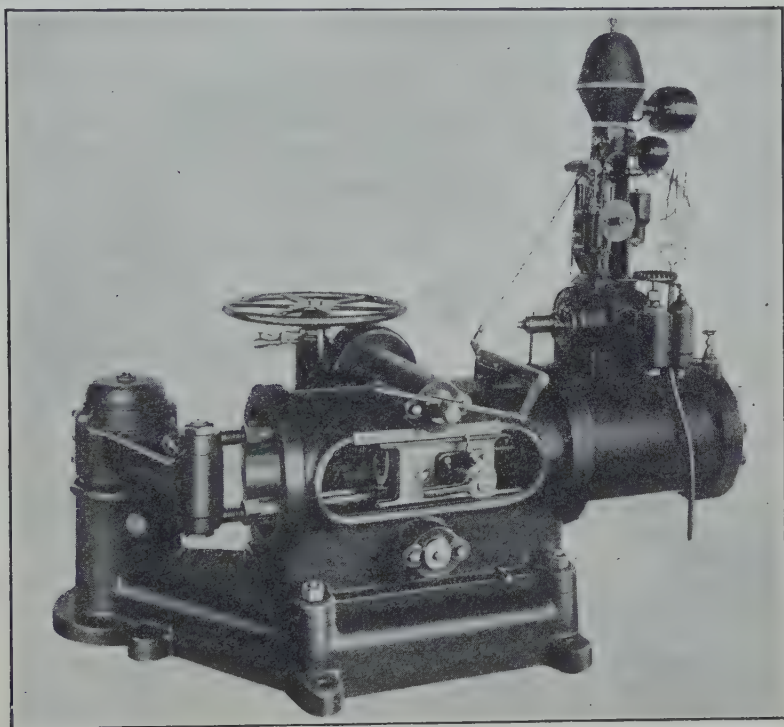


FIG. 237.—Woodward governor with motor-driven governor head—Woodward Governor Company.

method employs an electric motor to drive the fly balls, thus doing away with all pulleys, belts, and rods. Both of these methods have proved entirely satisfactory. They are a great convenience, since it is always difficult to work out an entirely satisfactory fly-ball drive from the main turbine shaft.

Governor Arrangements.—Very few entirely self-contained governors are now on the market. The smaller governors consist of two distinct parts and the larger ones of three distinct and separate parts.

In the case of the smaller governors, the fly balls, relay apparatus, compensating device, and regulating valves and cylinder are all mounted

together on the so-called governor stand, and this forms one distinct part, the oil-pressure pump and tanks forming the other distinct part. Most oil pumps are now motor driven in order to eliminate the troublesome belt and gear drives between the main turbine shaft and pump.

With the larger governors, the regulating cylinder is divorced from the governor stand and mounted directly above the turbine gates, thus eliminating heavy arms, shafts, etc., and reducing lost motion to a minimum. Frequently two regulating cylinders are employed for large turbines. The arrangement of the Woodward type HR governor with motor-driven head is shown in Fig. 237.

Oil-pressure Systems.—The central oil-pressure system for the governor oil supply in plants having a number of units is not now used so frequently as it was a few years ago. Such a system consists of two or three large oil pumps centrally located in the plant, any one or two of the pumps being of sufficient capacity to supply oil to all the governors, the remaining pump being held in reserve. This requires large and extensive piping and the space for it and often necessitates special construction. A single pump and tank equipment for each governor is now more frequently employed and found entirely satisfactory and less expensive than the central system and gives the same flexibility as a central governor system, when interconnected by piping and provided with one spare pump.

Performance Characteristics.—The important performance characteristics of a governor are:

a. Sensitiveness; that is, the minimum change in speed at any turbine-gate opening which will cause a change of governor position.

b. Stability; that is, the time required between a change of load and the settling down of the turbine to its normal speed with the new load.

c. Hunting or racing; that is, the periodic speed variation with steady load. A slight periodic speed variation, not in excess of one-half of 1 per cent, is not objectionable.

d. Capacity; that is, the work in foot-pounds which the regulating cylinder is designed to perform with any given oil pressure; also the capacity of the oil pump in gallons per minute and the volume of the pressure and receiving tank.

e. Speed regulation; that is, the speed variations which occur with sudden application or release of full or partial load. The results are generally expressed in the form of a ratio.

f. Regulating time; that is, the time in which a complete stroke of the regulating cylinder is made, for both closing and opening the turbine gates, and on which the speed regulation is based. Most governors are designed so that the regulating time can be varied by throttling the regulating valve ports and so that the opening time can be made longer than the closing time or *vice versa*.

SPEED REGULATION

Dependent Factors.—Assuming the governor is properly designed and of sufficient capacity to easily do the work of operating the turbine gates, then the speed regulation is dependent upon the following:

1. Flywheel effect of the revolving elements of the turbine and equipment driven by it.
2. Variation in effective head acting on the turbine, usually called pressure variations.

Speed regulation is generally expressed in terms of a ratio and with reference to two extremes of load.

Thus the speed regulation between two loads is defined as the ratio of the difference between the extreme speeds and the arithmetical means of the extreme speeds. Thus if

$$\begin{array}{ll} n_2 = \text{r.p.m. for load } A \text{ (the higher speed)} \\ \text{and} & n_1 = \text{r.p.m. for load } B \text{ (the lower speed)} \end{array}$$

then

$$\text{Speed regulation between loads } A \text{ and } B = \frac{n_2 - n_1}{\frac{n_2 + n_1}{2}} = \frac{n_2 - n_1}{n}$$

where n is the arithmetical mean of the two extreme speeds. It is usual to express this ratio as a percentage.

Flywheel Effect.—Flywheel effect is the capacity of a rotating mass to store or supply energy. It will, therefore, have a tendency to maintain constant speed of the turbine while the governor adjusts the flow of water after a change of load. It is equal to the polar moment of inertia of a mass, that is, the weight multiplied by the square of the radius of gyration, and is usually expressed as pound-feet squared and represented by Wr^2 .

Without some flywheel effect, satisfactory speed regulation of hydraulic turbines would not be possible, and it follows that speed regulation can always be improved by additional flywheel effect regardless of other conditions.

Modern alternating-current generator rotors designed for hydraulic turbine speeds are usually built in the form of a common flywheel and often have sufficient flywheel effect to obtain satisfactory speed regulation. The flywheel effect of the rotating parts of the turbine is usually neglected in computations for speed regulation, but it sometimes is sufficient to deserve consideration, especially with impulse turbines.

If a separate flywheel proves necessary, its weight for a given moment of inertia depends upon the permissible peripheral speed. In designing a flywheel, the runaway speed (for full gate opening and friction load) of

the turbine must be considered. For an impulse turbine, the runaway speed will be about 90 per cent above normal, and for reaction turbines this will vary from about 60 to 90 per cent above normal, depending upon the type characteristic of the runner.

Pressure Variations.—Any sudden change in the flow of water in an open or closed conduit supplying a turbine causes a rise or drop in pressure and consequently changes the effective head acting on it. These pressure variations can never be entirely eliminated, but they can be reduced by a proper choice of velocities and in the case of closed conduits by the use of pressure regulators, surge tanks, etc.

It is evident from the above that if the governor opens or closes the turbine gates to vary the flow after a change of load, the head acting on the turbine will be decreased with the opening of the gates and increased with the closing of the gates, which to some extent counteracts the effect of the gate movement and will cause the gates to overtravel momentarily. The governor cannot come to a steady position in such a case until the pressure variations have ceased.

Open-flume Turbines.—Where turbines are installed in open flumes, and the velocity of approach does not exceed 2 to 3 ft. per second, pressure variations due to changes in load and consequent gate movements are so small that they may be neglected. Speed regulation in such cases depends entirely upon the flywheel effect of the rotating element, assuming that a governor with proper characteristics is used.

Neglecting pressure variations, the speed regulation due to any given flywheel effect can be computed by means of the formula

$$s = \frac{KPT}{n^2Wr^2} \quad (1)$$

where K = a constant = 800,000.

P = maximum horse power of the turbine.

T = governor time to open or close the gates, seconds.

n = normal speed of turbine, r.p.m.

Wr^2 = flywheel effect, pound-feet squared (r being the radius of gyration in feet).

v = velocity of a point at distance r from the axis.

s = speed variation expressed as a ratio.

The above formula may be derived as follows:

The energy of a rotating mass is $Mv^2/2$. If the velocity of the mass is reduced from v_2 to v_1 , it will lose energy amounting to

$$\frac{M(v_2^2 - v_1^2)}{2}$$

but

$$(v_2^2 - v_1^2) = (v_2 + v_1)(v_2 - v_1)$$

and
$$\text{Energy lost} = \frac{Mv(v_2 + v_1)}{2} \times \frac{(v_2 - v_1)}{v}$$

but
$$\frac{v_2 + v_1}{2} = \text{average velocity} = v$$

and
$$\frac{v_2 - v_1}{v} = \text{relative change in velocity} = s$$

Substituting v and s in the above equation for energy lost, we have

$$\text{Energy lost} = Mv^2s$$

and
$$\frac{M(v_2^2 - v_1^2)}{2} = Mv^2s$$

and
$$M = \frac{W}{g} \text{ and } v = \frac{2\pi rn}{60}$$

Therefore,
$$Mv^2s = \frac{W}{g} \left(\frac{4\pi^2 r^2 n^2 s}{60 \times 60} \right) = \text{energy lost}$$

if the peripheral velocity of the mass is reduced from v_2 to v_1 .

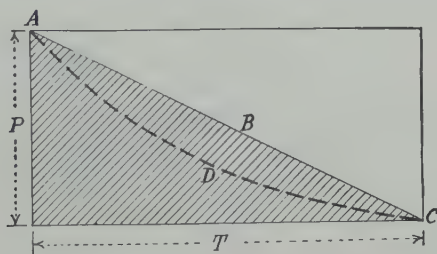


FIG. 238.—Time-power relation for flywheel effect.

Referring to the diagram, Fig. 238, let ordinates represent P = horse power, and abscissas represent T = governor time.

For a load change P , using curve ABC , Fig. 238, the total kinetic energy absorbed from the rotating mass will be $550PT/2$, which is equal to the energy lost if the peripheral velocity of the mass is reduced from v_2 to v_1 .

Equating, we have,
$$\frac{550PT}{2} = \frac{W}{g} \left(\frac{4\pi^2 r^2 n^2 s}{60 \times 60} \right)$$

and by substituting numerical values and transposing, we have

$$T = \frac{Wr^2n^2s}{800,000P} \quad (2)$$

and
$$s = \frac{800,000PT}{Wr^2n^2} \quad (3)$$

The true time-power relation more nearly accords with some curve like ADC , Fig. 238, due to the friction load of the turbine and generator,

which will vary with type of runner and size of generator as well as with the magnitude of the speed variations. Equation (3) is, therefore, approximate but gives safe values of s for both loads off and loads on.

The relation of the capacities of the regulating valve, regulating cylinder, and pressure tank will have a considerable effect on the relationship between the speed variations due to partial-load changes and those due to a full-load change. If the regulating cylinder is small compared with the valve and tank, then the regulating time will be nearly proportional to the movement of the regulating cylinder piston.

In actual practice, with commercial governors, T is nearly constant for all load changes, principally because the regulating valve does not open fully for the smaller load changes.

With modern commercial governors the relationship between part-load speed variations and full-load speed variations may be approximated as follows:

s effective = $s \times a$ where a has the following values:

100 per cent load change, $a = 1.00$

75 per cent load change, $a = 0.60$

50 per cent load change, $a = 0.30$

25 per cent load change, $a = 0.15$

Encased Turbines.—Most hydraulic turbines are installed in closed flumes or casings to which the water is carried under pressure in tunnels, pipe lines, penstocks, or other types of conduits, sometimes of considerable length.

Whenever the amount of water supplied to the turbine by means of such pressure conduits is varied by governor action, as in the case of a change of load, pressure variations are set up, depending in amount upon the head, length of conduit, and velocity of the water flowing in the conduit.

Since the head and length of conduit are fixed for any particular installation, the only direct opportunity to lessen the pressure variations is in a change of velocity.

Where the conduits are large or long, an effective change in velocity may result in excessive costs, and for such cases devices are resorted to which bring about the same result at less cost.

Surge tanks and pressure regulators are the most practicable of the various devices used to reduce pressure variation where the use of such devices is found to be less costly than a reduction in velocity to reach the same results.

In an economic study of such a problem, the value of the reduced friction head due to lower velocities in the conduits must be kept in mind.

The speed regulation of turbines subject to pressure variations is also dependent upon the flywheel effect, but, instead of a constant head acting

on the turbine, this will be variable with any change of load. This variation in head increases or decreases the amount of power which would be developed at the same gate opening for constant head, and the tendency of the flywheel effect to keep the speed constant is lessened in amount in proportion to the increase or decrease in power due to the variation in head from a constant quantity.

The calculated speed regulation s based on the flywheel effect is modified due to pressure variation approximately as follows:

For closing gates,

$$S = s(1 + p)^{3/2} \quad (4)$$

for opening gates,

$$S = \frac{s}{(1 - p)^{3/2}} \quad (5)$$

where s = speed variation, as a ratio, without considering pressure variations.

p = pressure variation expressed as a ratio of the pressure due to normal effective head.

S = speed variation as a ratio to normal, considering both flywheel effect and pressure variations.

Equation (1) shows that s is directly proportional to P (= horse power), the load, and inversely proportional to the flywheel effect Wr^2 .

If the head on the turbine is varied, the capacity varies as $\sqrt{H^3}$ (horse power varies as the square root of the head cubed). For an increased head = $(H + p)$, the capacity of the turbine will change with $(H + p)^{3/2}$; for a decreased head = $(H - p)$, the available energy will change with $1/(H - p)^{3/2}$.

Calling the original head H unity in order to obtain results as a ratio, we have:

For closing gates (load thrown off),

$$S = s(1 + p)^{3/2} \quad (6)$$

For opening gates (load thrown on),

$$S = \frac{s}{(1 - p)^{3/2}} \quad (7)$$

PRESSURE REGULATION¹

Pressure Variations.—Pressure variations are caused by changes in the velocity of a moving water column.

The maximum pressure rise in a pipe line would be the result of an instantaneous closing of the turbine gates or of a valve located near the turbine.

¹For a recent discussion of this subject see "Symposium on Water Hammer," Joint Committees of A.S.M.E. and A.S.C.E., 1933.

The moving water column represents an amount of energy which, upon closing of the turbine gates, will have to be dissipated in some manner.

The only means of dissipating the energy is by doing work in expanding the pipe and by compressing the water in the pipe.

The energy of the moving water in a section of the pipe line of any length l is

$$= \frac{w\pi d^2 l v^2}{8g} \quad (8)$$

where w = weight of water = 62.4 lb. per cubic foot.

d = inside diameter of pipe in feet, for an area a in square feet.

v = velocity of flow in pipe line.

The maximum force tending to compress the water in the pipe line is

$$\frac{wh\pi d^2}{4}$$

where h = pressure rise in feet. The average pressure will be half as great.

The distance along the pipe line which l ft. of water will be compressed is

$$\frac{whl}{K}$$

where K = modulus of elasticity of water in compression.

The work done in compressing l ft. of water is

$$\begin{aligned} \frac{wh\pi d^2}{4} \times \frac{1}{2} \times \frac{whl}{K} \\ = \frac{w^2 h^2 \pi l d^2}{8K} \end{aligned} \quad (9)$$

The maximum force in tension tending to burst a section of pipe of length l is

$$\frac{whld}{2}$$

The average force in tension is half as great.

The distance which the ring of steel forming the pipe will stretch is

$$\frac{whld\pi d}{2tE} = \frac{wh\pi d^2}{2tE}$$

where t = thickness of pipe wall in feet.

E = modulus of elasticity of pipe material in tension.

The work done in stretching the steel is

$$\frac{whld}{2} \times \frac{1}{2} \times \frac{wh\pi d^2}{2tE} = \frac{w^2 h^2 \pi l d^3}{8tE} \quad (10)$$

Setting Eq. (8) for the energy in the pipe equal to Eqs. (9) and (10) for the work done by the water in coming to rest gives

$$\frac{w\pi d^2 l v^2}{8g} = \frac{w^2 h^2 \pi l d^2}{8K} + \frac{w^2 h^2 \pi l d^3}{8tE}$$

Simplifying,

$$h^2 = \frac{v^2}{wg \left(\frac{1}{K} + \frac{d}{tE} \right)}$$

$$h = \sqrt{\frac{K}{wg} \frac{v^2}{\left(1 + \frac{Kd}{Et} \right)}} \quad (11)$$

Inserting the numerical values for $\sqrt{K/wg}$, where $K = 42.4$ mill. lb. per square foot,

$$h = \frac{145v}{\sqrt{1 + \frac{Kd}{Et}}} \quad (12)$$

For steel pipes this becomes

$$h = \frac{145v}{\sqrt{1 + 0.01 \frac{d}{t}}} \quad (13)$$

where $E = 4$ bill. lb. per square foot.

Equation (12) may be expressed as

$$h = \frac{v_p v}{g} \quad (14)$$

where

$$v_p = \frac{4660}{\sqrt{1 + \frac{Kd}{Et}}} = \text{velocity of pressure wave along the pipe line} \quad (15)$$

$$\text{(because } \frac{w.h.a}{2} = \frac{wal}{gT}v \quad \text{or} \quad h = 2\frac{L}{T}\frac{v}{g} = v_p \frac{v}{g})$$

as $2\frac{L}{T}$ is the velocity with which the pressure wave moves along the pipe.)

When the turbine gates are closed, vibrations or waves are set up in the water, the velocity of which depends upon the compressibility of the water, the elasticity of the pipe material, and the friction. This velocity of vibration can be computed by means of Eq. (15).

The effect of the velocity of vibrations upon the pressure variations depends upon the regulating time T , as will be seen from the following discussion.

Assuming a penstock of length L , with a basin at its upper end and a gate or gates at the turbine, the water with the gates open will have a velocity v . If the gates are closed, the velocity will decrease, setting up pressure variations and consequently vibrations. The vibrations will travel along the pipe line with a velocity a and will reach the basin in a time $t = L/v_p$.

They will be reflected from the basin and reach the gates in a time $t = 2L/v_p$, when the pressure will drop to normal and then below normal, and simultaneously a new pressure wave of equal magnitude but negative sign will pass up the pipe and be reflected from the reservoir, as was the positive-pressure wave, making the round trip in the same period of time. This alternate rise and fall of pressure will continue until damped out by friction.

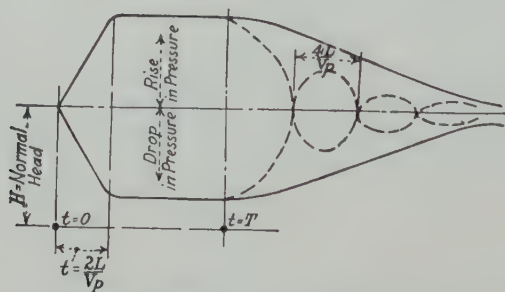


FIG. 239.—Effect of time on pressure wave.

New waves or vibrations will be set up for each increment of gate movement, and the pressure will rise until the reflected waves interfere, or up to the time $t = 2L/v_p$ (period $t = 0$ to $t = 2L/v_p$). Starting from the moment $t = 2L/v_p$, the pressure rise will remain constant since the vibrations will counteract each other.

When the gates are stopped ($t = T$), conditions will be different, depending upon whether they are fully or partially closed. If fully closed, fluctuations will occur having a period $4L/v_p$, but these will become smaller and smaller until normal pressure is reached.

If the gates are only partially closed in the time $t = T$, the pressure rise will decrease asymptotically, since the column of water still keeps moving, but with a changing velocity (see Fig. 239).

For any time less than $2L/v_p$, as no reflected waves have returned to the gates before they are closed, the maximum pressure will be the same as if the gates were closed instantaneously.

This time $t = 2L/v_p$ may, therefore, be termed the critical time, and it is evident that the governor-regulating time should be chosen appreciably greater than this critical time.

For all practical cases of turbine-speed regulation, the regulating time T will be chosen well in excess of $2L/v_p$, and for such cases the resulting pressure variations can be computed by the formula

$$h = \frac{NH}{2} \pm H\sqrt{\frac{N^2}{4} + N} \quad (16)^1$$

where h = pressure rise or drop, feet (beyond static head).

H = normal static head, feet.

$N = (Lv/gTH)^2$.

NOTE: N is the square of the pressure rise or fall, expressed as a ratio to H , for gate closure in time $T > 2L/v_p$; also $V = Q/a$, where Q is the

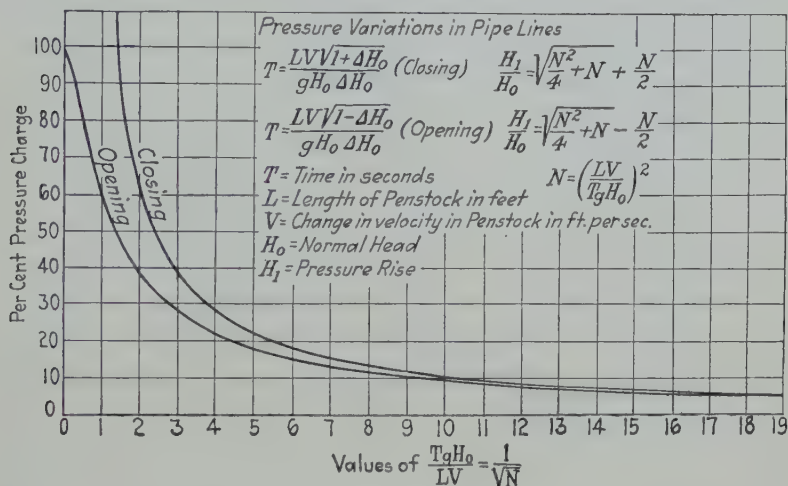


FIG. 240.

flow, in stopping, before the gate starts to close; in starting, after the gate is open and equilibrium established.

Use + for closing gates.

Use - for opening gates.

h from Eq. (16) expressed as a ratio of normal static head H is p in Eqs. (6) and (7).

The drop in pressure, due to opening the turbine gates, can never exceed H , the normal head, but approaches it as a limit.

Solutions of typical cases by means of Eq. (16) will show that for the same pipe-line conditions and the same governing time the pressure drop will always be less than the pressure rise.

Figure 239, furnished through courtesy of the S. Morgan Smith Company, will be found convenient for the quick solution of Eq. (16).

¹ Equation. (16) was derived by L. Allievi in a paper published in *Annali della Società degli Ingegner (Rome)*, vol. 17, 1902. See translation by Minton M. Warren, *Trans. A.S.C.E.*, vol. 79, pp. 250-255.

Solutions will also indicate that a small change in the penstock velocity will have an appreciable effect on the pressure variations. Hence, the velocity should be kept as low as may be consistent with a reasonable investment, keeping in mind that decreasing the penstock velocity will not only decrease the pressure variations and consequently improve the speed regulation but will reduce the friction-head loss as well.

If an economical solution of speed regulation cannot be found by changing the velocity, flywheel effect, and governing time, it is necessary to resort to the use of devices that will reduce the pressure variations, such as pressure regulators and surge tanks.

Pressure Regulators.—A pressure regulator is a device attached to the turbine casing or penstock near the turbine and operated from the governor in such a manner that, when the turbine gates are closed suddenly and a sufficient amount to disturb the regulation on account of pressure rise, the regulator will be opened sufficiently by the governor to keep the pressure rise within desired limits.

Such a pressure regulator should be adjustable so that the amount of pressure rise for any given load change can be varied for any set of conditions. It should close automatically and slowly under usual conditions so that its sudden closure will not produce a pressure rise. It should, moreover, be so arranged, that, if a sudden increase should occur after a sudden decrease in load during which the regulator is opened, the governor will close the regulator quickly and positively. Otherwise water then needed to prevent an excessive pressure drop would pass through the regulator while it was slowly closing.

If the discharge capacity of the pressure regulator is made large enough, the pressure rise can be reduced to a small amount even for a full-load change. Since it has no effect on the pressure drop due to opening the turbine gates, it would appear to have no value for speed regulation.

The load on a turbine is seldom increased suddenly or a great amount at a time, so that under practical conditions the pressure drop is never a serious matter as regards speed regulation.

Appreciable amounts of load are, however, frequently thrown off suddenly under usual operating conditions, and speed regulation is almost always improved by the use of a pressure regulator.

It is, moreover, always possible that the total load carried on the turbine at any time may suddenly be lost, as in the case of a short circuit or the tripping of an automatic circuit-breaker for various reasons, and it may become advisable to install a pressure regulator for the protection of the penstock or pipe line even if it is not essential to improve the speed regulation.

Various makes and types of pressure regulators are available. Figure 241 shows one of these, as made by the Allis-Chalmers Manufacturing

Company, used for both reaction turbines and impulse wheels for heads of from 300 to 2500 ft. Referring to Fig. 241, the outlet at the left, marked 40 in. in diameter, bolts directly to the turbine casing or nozzle pipe. The mushroom valve at the bottom, marked 27 in. in diameter, controls the outlet of the water, and its position is controlled by the pressure in the chamber just below the piston attached to the upper end of the shaft. The water-pressure supply either from the penstock or from a separate constant-pressure source is admitted through the opening marked Low Pressure Source. The rate of supply is regulated or throttled by the adjusting nut in the low-pressure-source elbow. The governor is connected to the slotted lever at the top of the illustration marked To Governor. When the gates of the turbines are closed, the motion of the relay rod through this slotted lever causes the regulating valve to open, which allows the pressure to escape from the underside of the piston and the pressure on the mushroom valve at the bottom of the regulator causes the shaft to move downward and allows water to escape from the casing. For quick movement, such as when the gates of the turbine are closing, the dashpot in the upper right of the illustration remains fixed, but, after the relief valve has opened, the dashpot slowly shifts its position and brings the regulating valve back to its closed position and the pressure again builds up under the piston and slowly brings the piston back to its closed position, its speed of closing depending on the setting of the adjustment on the water-supply source. Pressure regulators practically identical with these are in use on the reaction turbines at the Kerckhoff plant of the San Joaquin Light and Power Corporation Kern River plant, and the Pit River 1 development of the Pacific Gas and Electric Company, as well as on the impulse wheels at Big Creek 1, Big Creek 2, Western States Power, Eldorado, plants, etc., and are also used in connection with the impulse wheels for Kings River and Big Creek 2-A extension.

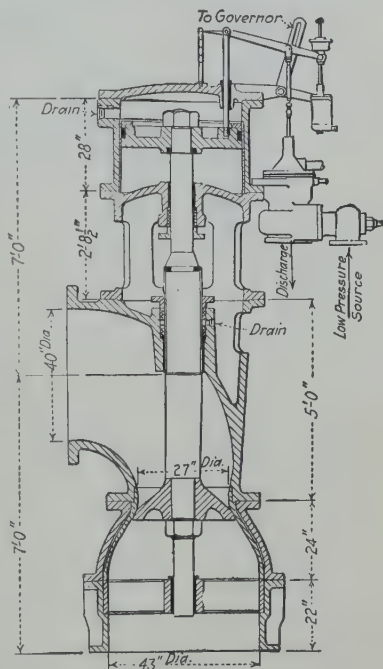


FIG. 241.—Pressure regulator—Allis-Chalmers Manufacturing Company.

Equalizing Reservoirs and Surge Tanks.—Since under practical conditions the pressure drop is always less than the pressure rise, the

latter can be kept within workable limits by means of the pressure regulator and satisfactory speed regulation maintained so long as the pressure drop is not excessive.

When conditions are such that the pressure drop becomes excessive for satisfactory speed regulation, the equalizing reservoir or a surge tank must be resorted to.

The results due to either are the same, namely, to reduce the effective length of the penstock and to supply or store up water during a change of load, while the flow of the water in the pipe line is accelerated or retarded.

The surge tank may be necessary to protect the penstock from dangerous pressure rises in case the pressure regulator should fail to act for some reason and will improve the speed regulation for small load decreases when the pressure regulator would not act owing to its insensitiveness. This insensitiveness may be increased when a surge tank is used so that the pressure regulator need not function so frequently, thus avoiding a waste of water.

(In this discussion that part of the conduit supplying water to the turbine located between the surge tank or equalizing reservoir and the turbine is referred to as the penstock, and that portion of the conduit located between the surge tank and the primary intake is referred to as the pipe line. This latter portion of the conduit is frequently a tunnel or combination of tunnel and pipe line.)

In plants using small amounts of water, the equalizing reservoir is seldom used, as the surge tank almost always proves the more economical. In large plants, operating under medium heads, where the quantity of water used is considerable, the equalizing reservoir will usually prove more economical in first cost and of much greater benefit to pressure variations and, hence, to speed regulation. The larger the surface area of the reservoir or surge tank, the smaller will be the pressure variations.

The regulating reservoir or surge tank should obviously be located as near the turbine as possible without incurring excessive cost. If arranged with an overflow, the pressure rise can be practically eliminated so that regulating reservoirs are usually provided with a spillway.

It is seldom satisfactory to arrange a surge tank with an overflow and usually not economical to do so. Surge tanks are almost always built high enough so that the water cannot overflow, even with a full-load change. Where topography permits, the surge tank may be placed on the surface of the ground, above the penstock line, at the point where the latter drops rapidly to the power house, as described for the Fagundes plant (Chap. VI, page 354).

In cold climates where there is danger of freezing, surge tanks must be well lagged or housed in, and in some cases heating within the housing must be restored to, to keep the surface from freezing over to such an

extent that the tank will not function. The use of an electric heater floating in the riser pipe of a differential surge tank has been found to be effective.

The minimum height of a surge tank is determined by the restriction that in no case must the water level in it drop to such a point that air is drawn into the penstock.

For practical purposes the necessary surge-tank dimensions can easily be calculated.

If the penstock is comparatively short and load is suddenly thrown on or off, a known quantity of water will be drawn out of the surge tank or stored in it and this will represent a certain definite amount of work. The quantity of water coming into the surge tank depends upon the

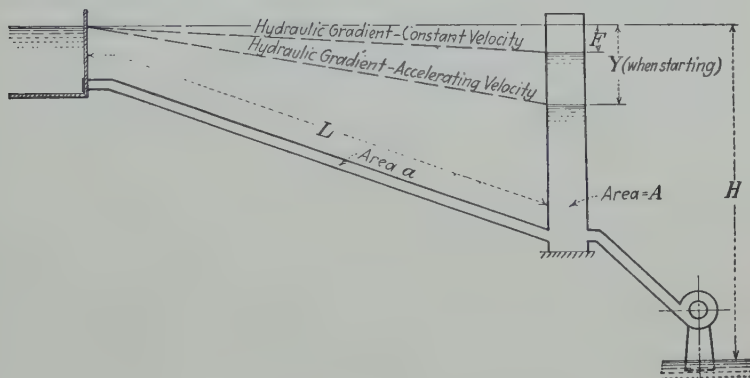


FIG. 242.—Penstock with surge tank.

difference in head between the surge-tank water level and the water level at the intake to the pipe line, and the work done in starting or stopping the water equals its kinetic energy. Neglecting friction, the work done in the surge tank would be (see Fig. 242)

$$\frac{1}{2}wAY^2$$

where w = weight of water = 62.4 lb. per cubic foot.

A = area of surge tank in square feet.

Y = maximum surge, up or down, in feet. In starting, Y is measured from the reservoir level; in stopping, it is measured from a distance below this, equal to the friction head.

The equivalent energy destroyed or gained in the pipe line would be

$$\frac{waLV^2}{2g}$$

where w = same as above.

a = conduit area in square feet.

L = length of pipe line in feet.

V = change in velocity, due to load change, in feet per second.

g = acceleration of gravity = 32.2 ft. per second.

Equating these,

$$\frac{1}{2}wAY^2 = \frac{1}{2} \frac{waLV^2}{g}$$

$$Y = \sqrt{\frac{aLV^2}{gA}} = Q\sqrt{\frac{L}{gAa}} \quad (17)$$

where Q = flow through penstock in cubic feet per second; in stopping, before gates start to close; in starting, after gates are opened.

Friction, however, enters into the problem. It will tend to hold back the water column in the conduit by delaying the acceleration of its velocity, thus increasing the drop in surge-tank water level. When the water rises, the frictional force will again retard the velocity change, thus reducing the maximum rise in surge-tank water level. If no additional change in load occurs, the water level will fluctuate with smaller and smaller oscillations, until finally a new static level has been attained.

If no friction prevailed, a sudden load increase demanding more water would draw down the level in the tank until the turbine gates were opened sufficiently to supply the demand under the reduced head. When finally the velocity in the pipe line had been accelerated sufficiently to stop the drawdown of the water level in the tank, the water would rise in it until the original level had been reached. The intermittent drop and rise of water level would not be damped out, and regulation would be impossible.

When friction is considered in arriving at the maximum surge, the mathematical solutions become impossible without some approximations.

Tests on existing plants with surge tanks show that Eq. (18) as written below gives results which are very nearly accurate.

$$Y = \sqrt{\frac{aLV^2}{gA} + F^2} \quad (18)$$

where F = friction head in pipe line between reservoir and surge tank in feet with cubic feet per second = Q

= CV^2 where C = a coefficient of friction expressed as the ratio of the total frictional head to the square of the velocity of flow.

The time interval which elapses between the turbine load change and the occurrence of the maximum surge can be computed by the following formula:

$$T = \frac{\pi}{2\sqrt{g}} \sqrt{\frac{LA}{a} + \frac{C^2V^2A^2}{a^2}} \quad (19)$$

Differential Surge Tanks.—Where wood-stave conduits are used for pipe lines, the friction loss is sometimes very small, and in such cases it may become necessary to introduce a choking effect between the pipe line and surge tank, thus increasing the effective friction on the surge-tank fluctuations.

This same effect is obtained by the use of the differential surge tank. The differential surge tank consists of a standpipe of about the same diameter as the conduit, freely connected to it, and a storage tank of larger diameter surrounding the standpipe and connected to it by a restricted passage. This restricted passage offers a resistance to the flow of water back and forth through it and reduces the surges.

In a simple surge tank, the level of the stored water, following a demand for more power, represents the accelerating level which is urging more water from the reservoir and determines the head acting on the turbine.

In the differential surge tank, owing to the resistance interposed between tank and conduit, the level of the stored water in the larger tank is independent of the accelerating head and the head acting on the turbine, both of these heads being determined by the water level in the small inner standpipe or riser. The riser acts like a simple tank of small dimensions which is supplemented by the inflow of the stored water from the larger tank in an independent variable quantity, thus having a tendency to break up any constant fluctuation by its non-synchronous action.

The internal standpipe or riser also affords a relief overflow into the larger tank in case of a sudden shutdown which causes the water to rise to the top of the standpipe, and, while spilling into the tank, the conduit receives the benefit of this back pressure and decelerates the velocity more rapidly, thus reducing the amount of water to be stored in the tank.

This type of tank can be built somewhat smaller than a simple tank for equal results and has the advantage that it will definitely prevent increasing surges under all conditions.¹

The size of the outside tank may be closely approximated as one-half the area of a simple surge tank, determining the latter by Eq. (18). The inside riser pipe is commonly about the same diameter as the penstock.

Figure 243 shows the differential surge tank as recently constructed at the Mystic Lake development of the Montana Power Company. The outer tank is of plate steel, $\frac{7}{16}$ to $\frac{1}{4}$ in., 12 ft. in internal diameter and about 118 ft. high, with a 12-in. spiral-pipe overflow (not shown in Fig. 243). The riser pipe is 44 in. in diameter, with port area 2.0 sq. ft. The tank is located at the lower end of a 56-in. wood-stave pipe line about

¹ For a complete description, with theory, of the differential surge tank, see Raymond D. Johnson, *Trans. A.S.M.E.*, vol. 30, 1908, p. 443, and *Trans. A.S.C.E.*, vol. 78, 1915, pp. 760-803.

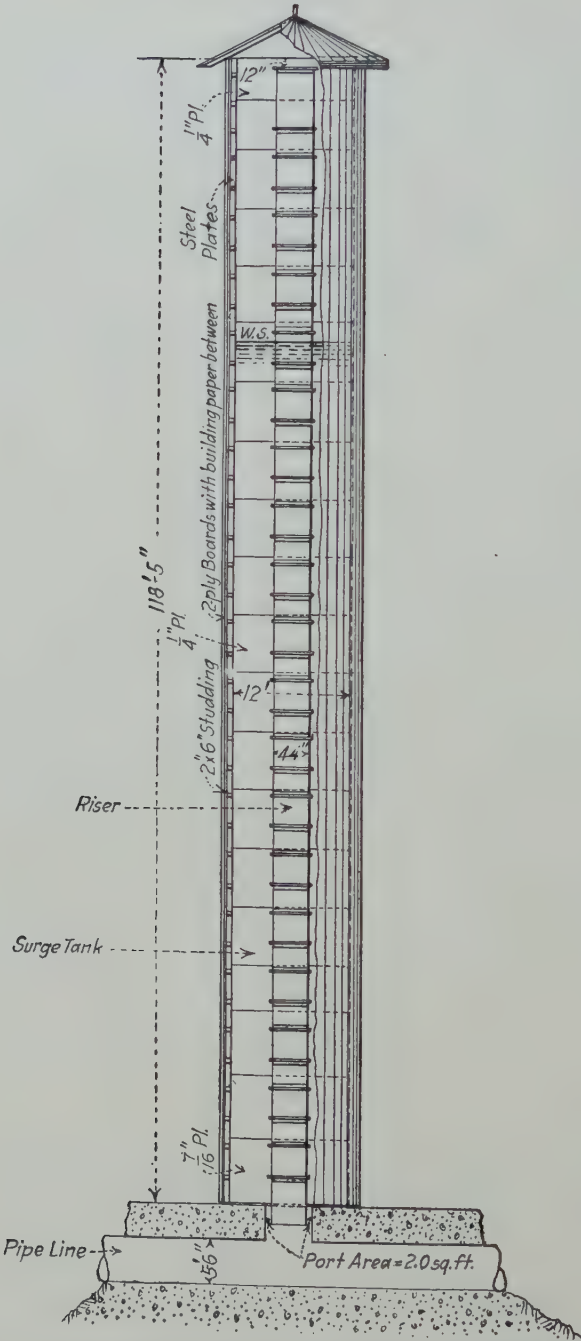


FIG. 243.—Differential surge tank—Mystic Lake development, Montana Power Company.

9000 ft. long, which in turn leads from a 5- by 7-ft. rock tunnel 1000 ft. long. The steel tank has a wooden covering and roof for protection against frost. Between surge tank and power house, about 2732 ft., a welded-steel penstock varying in diameter from 48 to 28 in. descends about 1025 ft. to two 7500-hp. Pelton wheel units operating at 300 r.p.m. under a head of 1050 ft. (see Chap. XIII, Fig. 299).

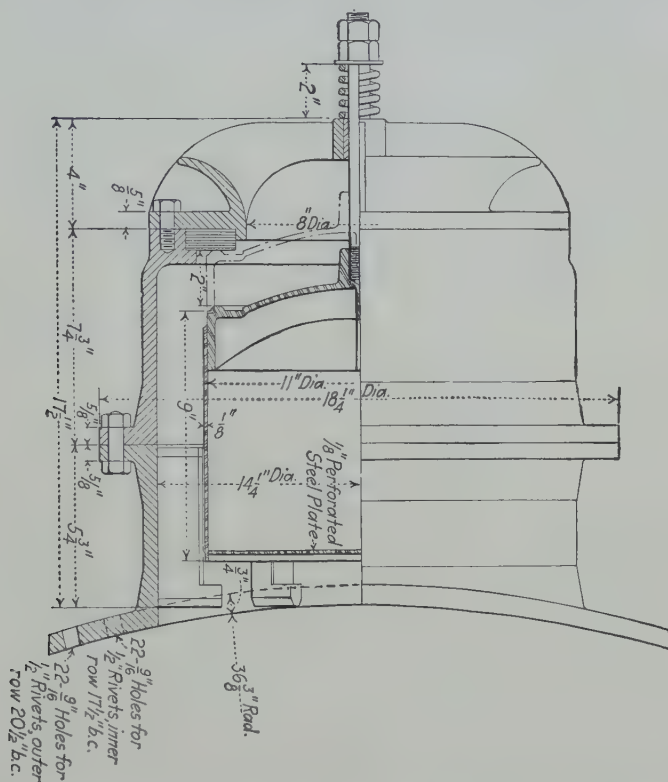


FIG. 244.—Eight-inch air inlet valve for 6-ft. steel pipe line—Coffin Valve Company, Boston.

The largest differential surge tank thus far built is at the Walenpaupach development of the Pennsylvania Power and Light Company near Hawley, Pa.,¹ and is of plate-steel construction 55 ft. in diameter and 135 ft. high, with a capacity of 2.4 mill. gal. The riser pipe is 8 ft. in diameter and extends to within 3 ft. of the top of the surge tank. This tank regulates a conduit 18,000 ft. long and 14 ft. in diameter and provides for instantaneous rejection of full load without spilling. Full-load velocity is reached in 3 min., and likewise, for full load rejected, water in the conduit is brought to rest 3 min. after the turbine gates are closed.

¹ Report of Hydraulic Power Committee, Nat. Electric Light Assoc., pp. 1121-1122, 1925.

There are two 28,500-hp. wheel units operating at 300 r.p.m. under a head of 330 ft., and this station operates as an automatic one, which is unusual with such large units.

Air-inlet Valves.—The profile of the penstock line is often such that the lower portion near the wheels has a much steeper slope than the remainder of the line. Under these conditions, if complete load acceptance occurs rapidly, the water in the lower portion of the penstock may accelerate quicker than that in the upper part, and there will be a tendency for the column of water to separate at the change in grade and subject this part of the pipe to a collapsing pressure.

This difficulty is removed by a surge tank at or near the part of grade change. Where a surge tank is not provided, an air-inlet valve to allow

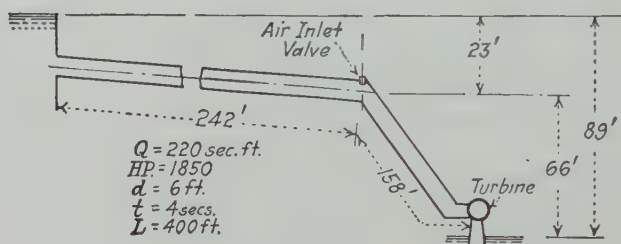


FIG. 245.

air to enter the pipe as needed will provide a means of avoiding collapsing pressure on the pipe.

Such an air-inlet valve is simply a check valve set in the top of the pipe, normally held shut by the penstock pressure and also aided in this respect by a light spring, so that the valve will not operate and waste water for slight pressure differences. Figure 244 shows an 8-in. air-inlet valve as made by the Coffin Valve Company.

To determine the necessary size of air valve the following method may be used, conditions being assumed as shown in Fig. 245 and complete load acceptance of 1850 hp. assumed to occur in 4 sec., which corresponds to a change in velocity from 0 to 7.9 ft. per second. The accelerating head h_a required for this change of velocity may be found by use of the relation $F = M\Delta v$ or

$$wAh_a = \frac{wAL}{g} \frac{\Delta v}{t}$$

whence

$$h_a = \frac{L\Delta v}{gt}$$

whence

$$h_a = \frac{400 \times 7.9}{32.2 \times 4} = 24.5 \text{ ft.}$$

for the penstock as a whole.

Actually, considering the two sections formed by the break in grade and assuming h_a for each section to be proportional to the head on the section as compared to the total head of 89 ft.,

For the flatter section:

$$24.5 \times \frac{23}{89} = 6.4 = \frac{242 \times V_1}{32.2 \times 4}; \quad V_1 = 3.4$$

For the steeper section:

$$24.5 \times \frac{66}{89} = 18.1 = \frac{158 \times V_2}{32.2 \times 4}; \quad V_2 = 14.7$$

The two water columns would, therefore, tend to separate at a rate of $(14.7 - 3.4 = 11.3)$ ft. per second, which would require $11.3 \times 28 = 317$ cu. ft. of air per second to replace water with air.

Allowing a maximum difference of pressure of 2 lb. per square inch between the outside and inside of the pipe and assuming a coefficient of 0.85 for air entrance:

$$317 = 0.85 a_v \sqrt{2g \times 2 \times \frac{144}{0.0764}}$$

where a_v = area of air valve in square feet

and 0.0764 = weight in pounds per cubic foot of air (at 60° F.)

whence $a_v = 0.76$ sq. ft.

or a 12-in. air valve would be required.

This will require a velocity through the air valve of

$$\frac{317}{0.78} = 406 \text{ ft. per second.}$$

It is essential that an air valve be set in a manhole or protective covering in such a manner that free access of air will be provided at all times and it must also be protected from frost so that it will be available for action at any time.

CHAPTER XI

TRANSMISSION LINES

ECONOMIC FEATURES OF ELECTRICAL TRANSMISSION

The following pages describe the three phases of the growth of electrical transmission illustrated by examples; enumerate the values of interconnection, first to an all-steam system and then to a mixed steam and hydroelectric system; and finally discuss the characteristics of very high tension transmission and the probable maximum distance and voltage which may be attained in the future.

Results of Progress in Electrical Transmission.—The growth in power development during the past 35 years is due primarily to progress made in electrical transmission, with a steady increase in line voltage, and longer, more economical, and reliable transmission over longer distances.

This advancement has proceeded in three stages:

1. The development and transmission of water power with the output of remote hydroelectric plants available at load centers. Power is now transmitted 250 miles or more at 220,000 volts. [Transmission from Boulder dam to Los Angeles (265 miles) will be at 280,000 to 330,000 volts at sending end.]

2. With reliable transmission small steam plants are being replaced by a few large and more economical plants.

3. As a result of the foregoing, the advantages of interconnection of load centers of different power systems—particularly those using water power—became apparent and this has proceeded rapidly. This interconnection has usually begun on a small scale, often where the transmission line of a company ran close to a load center of another system or crossed its lines, permitting interchange of power to the benefit of both companies.

Advantages of Interconnection.—The interconnection of generating stations is creating new values in the following seven ways:

1. By making available at the load centers water power from large and remote, low-cost developments, as is done with the systems of the Southern California Edison Company and the Pacific Gas and Electric Company.

2. By increasing the firmness or permanence of the water power, the possible results depending on the character and location of the interconnected plants and the diversity of the stream flow. A maximum value is reached where surplus or "run-of-stream" plants are interconnected with plants controlling storage on the stream above, in such

amounts that the resultant combined output is made largely or wholly permanent. Thus the Deerfield River system of the New England Power Company, where the Davis Bridge (Harriman) plant controls the flow from reservoirs of about 7.5 bill. cu. ft. capacity, can be operated in conjunction with the other plants of the system in such a manner as to produce practically uniform power.

3. By making possible a larger permanent capacity value of "run-of-stream" plants in a mixed steam and hydroelectric system where the water power, if good pondage is available, can be used on the peak of the load and carry, at a low load factor, a capacity greatly in excess of what it would be capable of carrying as an isolated plant on a commercial load factor.

This is illustrated by the Conowingo water power development of the Philadelphia Electric Power Company of about 350,000-hp. capacity, which is tied in with about 500,000 kw. of steam power. When river flow is ample, the base load is carried by the water power; at low river flow, steam carries the base load and water power the peak, thus utilizing the capacity of both steam and water power in a much more effective manner than would be otherwise possible.

4. By decreasing the standby or reserve equipment, one of the most direct and self-evident of the economies afforded by interconnection. For example, where a system load has reached the capacity of its stations and additional capacity is required, assuming a system of 30,000 kw., it would be desirable to install at least a 15,000-kw. unit to obtain the economy in operation possible with larger sized units. This would mean carrying a 50 per cent reserve capacity when probably 25 per cent would be ample for the expected growth in load in the period considered. If an adjacent system had new capacity as much in excess of its needs, then a considerable saving to the first company might be effected by interconnection.

5. By increasing the load factor, which increase is proportional to the diversity of the connected load. An analysis of the load factor of isolated steam systems, in comparison with that of hydroelectric companies which usually depend upon interconnection to market their power, shows a decided margin in favor of the hydroelectric company. Much of this advantage, however, is due to the fact that a large amount of marginal or surplus output of a water power plant, sold at less than fuel cost, helps in raising the load factor of the hydroelectric system.

6. By increasing the radius of service from a given plant larger sizes in both unit and plant capacity have been made possible. Great economies are thus effected in the construction and operating costs of both water power and steam plants, particularly the latter, where the tendency is toward replacing small and isolated plants with units from 30,000 to 90,000-kw. capacity in plants of an ultimate size of 100,000 to 1 mill. kw.

7. By allowing the division of load between stations for most economical operation, with the result that the most efficient plants supplying a load center are now generally used for base-load generation only and the peak load is carried by the older plants.

Limits of Interconnection—Steam Power Systems.—The economies noted are effected when steam plants serving a power district can be constructed in large sizes and interconnected with the adjacent economical water powers in such a way that, on partial or complete failure of one of the plants, the load can be safely carried by the remaining ones. Such economies cease when any power introduced by further interconnection would cost more per kilowatt-hour than at the local steam plants. The most efficient plant, moreover, should be available for use in carrying the base load of the district. With this in mind it is theoretically possible, considering only steam-plant generation, to define the limits of interconnection.

First, it has been found that to obtain sufficient condensing water for large steam plants they must be located either at tidewater or upon large rivers. This has rendered it generally impossible to locate large steam plants at the mines and transmit power to load centers.

Second, there are in many cases older steam plants available for peak and reserve capacity without transmission from other load centers.

Third, load curves of different systems are often so similar that little advantage in decreasing load peaks and increasing load factor results from interconnection.

Fourth, the comparative saving of large plants rapidly decreases as the size of the smaller station increases.

A modern 30,000-kw. plant, such as would be required in serving a population of from 60,000 to 150,000 people would, therefore, not benefit greatly from interconnection under average present conditions, when it is considered that most of our cities of that size are farther apart than 10 to 30 miles, except in the northeastern section of the country. With the exception of this section it may be safely concluded that, excluding water power, no extensive interconnection will take place between load centers under present conditions of increasing economy in steam-plant design, where the adjoining load centers are equally favorably situated with regard to coal supply and condensing water.

It must be kept in mind, however, that the foregoing are only general conclusions, and specific cases may present variations. Thus plants may have special advantages in fuel supply or condenser-water supply which give them a large radius of service to communities not so favorably situated. When new steam capacity is required for load increment alone, interconnection may present a considerable capital saving if a large steam plant is available at an adjacent load center—subject, however, to a

well-grounded prejudice in favor of the local plant with its less likelihood of interruption in service.

The uncertainties of transmission may often be minimized by making several lines and sources of power available, but there must be a permanent daily flow or an annual interchange of power at a profit rather than the secondary value of interconnection in the form of breakdown or reserve service.

In the northeastern section of the country there are many large-load centers within 30 miles of each other and near enough to warrant interconnection, assuming that one has a marked advantage over its neighbor in plant size, location, or fuel supply, as is the case with the Montaup Electric Company. While more such centers will be interconnected when steam-plant production becomes sufficiently stabilized to make new reserve capacity a problem, even then the established centers where the demand reaches 100,000 kw. probably will not be benefited by interconnection, as the advisable increment of capacity required in anticipating growth of load will be sufficiently large to allow the use of a unit of 30,000-kw. capacity, beyond which size the gain in economy is relatively small.

This has been due (1) to the large size of the load centers like Boston, New York, or Philadelphia best fitted for independent operation, with large steam plants at tide-water; (2) comparatively short distances, not requiring high-voltage transmission; (3) lack of development of local water power still available, owing to high cost, and of more distant lower cost water power due to state boundary limitations and other difficulties, which are gradually being overcome by legislation and agreement. In 1930, however, the Lower Fifteen Mile Falls (Comerford) plant began operation upon the upper Connecticut River, transmitting power at 220 kilovolts to Tewksbury, Mass., and thence at 100 kilovolts to the Boston Edison Company system—a total distance of about 150 miles. Similarly, the Conowingo and Safe Harbor plants now supply the Philadelphia and Baltimore load centers through 220-kilovolt lines, and the tendency is toward the use of available water power in the large-load centers throughout the Northeast.

Interconnection—Water Power Predominant.—Referring again to transmission lines of over 100,000 volts, it is notable that where water power is the principal source of power, as on the Pacific coast and in the South, where water power predominates, the greatest progress has been made and it may be concluded that the first three of the seven advantages of interconnection previously stated are of greatest importance.

While, as previously shown, the load center under all-steam-plant generation is of limited radius, with mixed water power and steam generation the extent of interconnection is not usually limited, as the distance which the best permanent water power can be transmitted usually exceeds

the distance between large-load centers. This is resulting in a complete network of transmission wherever water power is extensively developed. The distance to which water power can be transmitted in competition with steam has, however, its limits.

Economic Limit of Transmission Distance at 220,000 Volts, 60 Cycles.

Power from large low-cost permanent water power projects from 100,000 to 1 mill. kw. or more in capacity, such as can be developed in eastern Canada and on the St. Lawrence River, can be transmitted at an average-system load factor 350 miles at a saving over best present steam-plant generation, which is sufficient to justify the construction of these plants. The base load can be transmitted even greater distances at a saving over best steam-plant generation on an equivalent load factor. E. L. Moreland, after a very careful study of a definite project, has declared it feasible to transmit such base loads a distance of 500 miles at 220,000 volts.¹ Higher voltages, however, would probably be used for these distances.

Surplus and Permanent Power.—It is important here to emphasize again, to avoid confusion, the fundamental difference between the two products of hydroelectric plants: surplus and permanent power. The value of permanent power, as discussed in Chap. XII, depends on the relative cost of the steam power produced at or transmitted to the market in question, which in the very large capacities mentioned here is now generated at an average cost and average load factor, including fixed charges, of about 1.0 ct. per kilowatt-hour.

Surplus power, on the other hand, on account of its intermittent nature, saves fuel only, and its value is ordinarily determined on that basis. There are two classes of surplus power: the first class might better be called secondary and is that output possible from the spare generating capacity not used for the system load and non-permanent in character; the second class is that marginal or by-product off-peak and night power resulting from the 3 or 4 months freshet flow, usually greatly in excess of the generating capacity, a part of which is sold in as large quantities as the market can absorb. Today, 5 mills per kilowatt-hour for daytime, or secondary power, and about 2 mills for the salable night surplus, or marginal power, are common figures comparable to the costs given above for permanent power.

With the rapidly increasing economy in fuel consumption in the base-load stations, the latter figure probably will become even less, perhaps as low as a mill per kilowatt-hour. It is doubtful, however, whether there will be any decrease in the value of the daytime secondary power, as there will always be a demand for this kind of power to use as peak capacity, for the reason that, when not too intermittent in character, it

¹ "Superpower Transmission," *Jour. A.I.E.E.*, February, 1924, pp. 1-103.

will save maintenance and operating as well as fuel costs. In a peak-load station these items are always considerably higher than for the base-load station, owing not only to the fluctuation of the load carried but also to the fundamental difference in the design, which tends toward a larger capital expenditure in the base-load station to obtain very efficient machines, operated at a high-load factor, while low first cost in the peak-load station is the principal consideration, as the fuel consumption is relatively small and a considerable variation in fuel cost affects but little the average cost of power.

As approximate values, 1 ct. per kilowatt-hour may be assumed for permanent power on an average load factor of 40 to 45 per cent, and 5 mills for secondary daytime power. Few water powers, even the largest and most favorable, can be developed at a cost much less than \$100 per kilowatt, which means with a commercial load factor a cost of about $3\frac{1}{2}$ mills per kilowatt-hour generated. The margin left for transmission of permanent power would be about 5 mills, allowing $1\frac{1}{2}$ mills for line losses and for a safe margin below steam costs to insure the sale of the power. This amount would allow at average load factor, as stated previously, about 350 miles of transmission. Returning to the question of the transmission of surplus power, it is evident that with a price of 5 mills and a cost of $3\frac{1}{2}$ mills, if only secondary and marginal power were produced, power developments of this type would not be attractive, as under these conditions there is no margin left for transmission after allowing for losses and terminal transmission equipment. As most of our large water powers have a considerable proportion of secondary power, their utilization becomes on the face of it, at any rate, a much more difficult problem.

To offset this low value, however, there are three considerations to be kept in mind:

First is the possibility, previously stated as the second advantage of interconnection, of developing "run-of-stream" power in conjunction with high- or medium-head developments with storage behind them in such a way as to change the nature of the output from surplus to permanent power, thus actually doubling its value. The plant with storage will of course usually cost more than the run-of-stream plant, but there are many opportunities of this sort that will show a very favorable return on the investment.

Second, in considering the cost of secondary power, it is frequently the case that where a low-head plant requires practically no outlay for waterways, such as at a concentrated fall where the power house is built as an integral part of the dam, the development can be increased in capacity at small additional cost, sometimes for \$50 per kilowatt or less. Further analysis, however, indicates that the kilowatt-hour cost of the additional power is little changed, inasmuch as the added power output

is usually available only for less than 50 per cent of the time, which partly counterbalances the reduction in cost.

Third is the high value of permanent water power as capacity when used on the peak of the load at ordinary or low-water stages in a mixed steam and hydroelectric system (noted previously as the third advantage of interconnection). Thus, the top or peak 40 per cent of capacity of the entire northeastern section in 1919 carried only 7 per cent of the total annual output, equivalent to a load factor of only 7 per cent as compared to the 41 per cent average load factor for the whole region. On the day of maximum peak the same 40 per cent peak capacity operates on a load factor of only 28 per cent. For this controlling condition the water by pondage will produce three and one-half times as many kilowatts as it would under continuous operation and nearly 50 per cent more than it would at an average load factor. For example, if a 100,000-kw. plant serving in a combined steam and hydroelectric system with a steam plant of 150,000-kw. was designed for 100,000 kw. at 40 per cent load factor with, say, 70,000-kw. permanent water power at that load factor and 30,000-kw. surplus, the power could safely be considered wholly primary, as on the peak-load day the water sufficient to develop 70,000 kw. would carry by pondage the 100,000-kw. peak at the lower load factor.

Secondary power, therefore, can usually be developed at a low cost. With combined steam and water generation, where pondage permits the use of water power to carry the peak load or where natural and artificial flow diversity is provided by interconnection, the secondary power, in fact, may become primary in nature.

Probable Maximum Economic Limit of Transmission Distance.—The maximum economic distance of high-tension transmission depends primarily on the difference in annual cost between local steam and water power production at 100 per cent load factor. This difference, capitalized, determines the maximum economic length of transmission line. There should be a sufficient margin, however, to enable the water power to sell at a cost slightly lower than local steam and to compensate for the additional risk involved over ordinary steam-plant construction. As steam-generated power costs are being steadily decreased, the amount which can be spent for transmission becomes smaller and smaller. Based on present knowledge and on the assumption that the probable lower unit costs of transmission line at voltages higher than 220 kv. will be offset by the increased cost of terminal equipment, it seems scarcely probable that this difference in cost between steam and water will make it possible for power to be transmitted on an economic basis over 600 miles to our larger northeastern load center. There are today, however, few places in North America where large permanent water powers are found farther than this distance from a market sufficiently large to absorb them. Most

of these are in Canada, particularly in northern Quebec, where the increased cost of development due to lack of transportation facilities considerably reduces the economic transmission distance, and by the time the country is developed to such an extent that these powers can be built at a reasonable cost, the local market is likely to be developed so that the power can be utilized nearer its source.

Few water powers have the three necessary qualifications of large size, low cost, and permanence of output in such a degree as to make it possible to transmit power any such distance as that of 600 miles stated as the economic limit. This limit may never be reached for the added reason that the demand within the 600-mile radius of the plant in question may completely absorb the power.

It is interesting to note that in the southeastern section it is possible to transmit at 100,000 volts over a distance of approximately 500 miles, from Raleigh, N. C., to Birmingham, Ala. Although at that voltage there would be no economic justification for doing so continuously, on account of excessive line losses, yet a large block of power for a considerable period was successively transferred from the Muscle Shoals steam plant at Sheffield on the Tennessee River in the extreme northwestern part of Alabama over the systems of the Alabama Power Company, the Georgia Railway and Power Company, and the Southern Power Company to the system of the Carolina Power and Light Company at Raleigh, which enabled the latter company to maintain its services in a period of extreme low water and to fulfill its obligations to its customers. This is sending power beyond the economic kilowatt-hour limit on which the maximum transmission distance of 600 miles was based, as the cost per kilowatt-hour for this power was much greater than that for which it could have been produced at Raleigh, had there been capacity available there. Power in the future, as higher voltages are interconnected, may be transmitted much greater distances in emergencies but not on an economic kilowatt-hour basis.

Characteristics of Extreme High-tension Transmission.—The growth of voltage has been almost uniformly at the rate of about 6000 volts annually for the last 35 years, and it is almost certain that the present maximum of 330,000 volts by no means marks the limit if the use of 60 cycles is continued for such high-voltage transmission. Electrical engineers believe that the maximum voltage will be restricted by economic reasons rather than insurmountable engineering difficulties. That this opinion is based on practice, as well as theory, is brought out by the fact that laboratory investigations have already been successfully made at 1.5 mill. volts.

The proper voltage for any given frequency, distance, and quantity of power of a transmission line depends on balancing of greater line losses and line cost per kilowatt inherent in the lower voltage against the greater

cost of the terminal equipment, the transformers, switches, synchronous condensers, etc., at the higher voltage.

The question may be determined solely by the charging capacity in kilovolt-amperes necessary at the sending end, to bring the circuit up to voltage, which in extremely long-distance, high-voltage transmission may involve an amount in excess of the generating capacity.

The desirability from an electrical standpoint of having each circuit operated as a unit with respect to generators, transformers, and switches, for the purpose of protection against short circuit, was noted by F. W. Peek, Jr.,¹ as a reason for effectively limiting the voltage, as the largest generators and transformers which can be built and shipped as a unit are now being approached in size. As equipment is increased in size, however, the unit cost decreases and the efficiency increases so that, as the need arises, it is likely that this difficulty may be overcome.

The cost per mile of the line increases with the voltage principally for the reason that the conductor must be made larger in diameter to prevent corona, which is the phenomenon and consequent loss of power caused by the rupturing of the dielectric strength of the air around the conductor. For a given wire, material, and strand, the construction cost of a line of any particular type and voltage is fixed, as almost all quantities involved depend directly upon the diameter and upon the maximum working stress in the conductor. As the stress is a function of the square of the diameter, the desirability of having a diameter large enough to be free from corona and its accompanying losses is frequently the controlling factor in the cost of any given voltage transmission line above 110,000 volts.

The reactance of a long line carrying high voltage is also so great that capacity in the form of synchronous condensers must be supplied in order to control the voltage or, in other words, to prevent poor regulation, which would otherwise result without such provision. The control of voltage is necessary at both ends of such a line, and the advantage in providing condensers in the middle as well as at the receiving end of lines involving great length was shown in a paper by P. H. Thomas.²

There is also the serious electrical problem of stability or the ability to maintain voltage under load and to regain a steady condition after interruption. This results in reducing the capacity that can be delivered for a given distance at any given voltage and is important in that the economy in high-tension transmission lies in increasing the amount of power that can be transmitted in a given circuit. T. A. Worcester² estimated that the cost on an 82 per cent load factor, with 80,000-kw. capacity, assumed as a stable amount under worst conditions, would be

¹ *Trans. A.S.C.E.*, vol. 86, pp. 757-771, 1923.

² "Characteristics of a 500-mile, 60-cycle, 220,000-volt Transmission Line," *Jour. A.I.E.E.*, 1924, pp. 1-103.

6.7 mills per kilowatt-hour assuming very low cost water power generation. The equivalent cost of steam is slightly less than this figure, and the conclusion may be reached that in this case stability effectively limits the economic distance to below 500 miles. However, a higher value of the stable power as estimated by some engineers would result in increasing the economic distance.

Judged by the distances practically attained by lower transmission voltages and the empirical rule that the kilovolts of a line should equal the distance in miles under usual circumstances, this distance of 500 miles would seem better adapted to higher than 220,000 volts transmission. Mr. Worcester¹ further states that such an increase in voltage would allow an increase in stable power and that the increase in cost would just about balance the increase in power, but by going to 25 cycles, using a frequency changer at the receiving end and allowing 10 per cent for the additional loss in conversion, a 500-mile 220,000-volt line would be able to carry 150,000 kw. per circuit and operate with all the certainty of the present 60-cycle 220,000-volt 200-mile lines and at a cost for 135,000-kw. capacity of 10 per cent less than given above for 60-cycle transmission, or about 6 mills per kilowatt-hour. This would leave a margin of nearly 10 per cent over steam costs and confirms the possibility previously stated of economic transmission at 600 miles and 100 per cent load factor, leaving the same margin under steam costs of 10 per cent in both cases.

As there is a decided gain in transmission at 25 cycles over 60 cycles, enough to more than counterbalance the extra cost and loss involved in frequency changers as shown in the preceding paragraph, mention should also be made of the possibility of using direct current, which would still further simplify the transmission problem, as reactance would be eliminated and corona considerably reduced. The mercury-arc rectifier, which may yet transform high-voltage alternating current to direct current, is a possibility of the future and is now used at much lower voltages successfully and on a commercial scale, both in this country and in Europe.

ELECTRICAL FEATURES OF TRANSMISSION

Notation

E_d = difference in potential or voltage drop between two points on the conductor, measured in volts.

I = current flowing in conductor, measured in amperes.

R = resistance of conductor, measured in ohms.

W = power in watts equal to $\frac{1}{1000}$ kw.

W_l = power lost in line in watts equal to $\frac{1}{1000}$ kw.

E_n = voltage to neutral or voltage in each phase.

E = line voltage or voltage between wires.

f = frequency in cycles per second.

L = inductance in henrys.

C = capacity in farads.

¹ "Characteristics of a 500-mile, 60-cycle, 220,000-volt Transmission Line," *Jour. A.I.E.E.*, 1924, pp. 1-103.

E_r = line voltage received, or load voltage.

E_s = line voltage sent, or generated voltage.

X = reactance, effect of capacity and inductance on circuit, in ohms.

Z = impedance, effect of resistance, capacity, and inductance on circuit, in ohms.

ϕ = angle of lag or lead, or angle of phase displacement.

$\cos \phi$ = power factor or ratio of effective voltage (in phase with current) to the impressed voltage.

d = diameter of wire in inches.

T = temperature rise in degrees centigrade.

D = equivalent equilateral spacing (see footnote, Table 93).

Basic Equations for Three-phase, Three-wire Transmission

$$E_d = IR \text{ per phase} \quad (1)$$

$$W_l = E_d I \text{ per phase} \quad (2)$$

$$= 3E_d I \text{ per circuit}$$

$$W_l = I^2 R \text{ per phase} \quad (3)$$

$$= 3I^2 R \text{ per circuit}$$

$$E = \sqrt{3}E_n \quad (4)$$

$$\left. \begin{aligned} W &= 3 E_n I \cos \phi \\ &= \sqrt{3} E I \cos \phi \end{aligned} \right\} = 1000 \text{ kw.} \quad (5)$$

$$\left. \begin{aligned} \frac{W}{\cos \phi} &= \frac{3 E_n I}{\cos \phi} \\ &= \sqrt{3} E I \end{aligned} \right\} = \frac{1000 \text{ kw.}}{\cos \phi} = 1000 \text{ kva.} \quad (6)$$

$$E_{ns} = \sqrt{(E_{nr} \cos \phi + IR)^2 + (E_{nr} \sin \phi + IX)^2} \quad (7)$$

$$\text{Regulation} = \frac{E_s - E_r}{E_r} \quad (8)$$

$$\cos \phi' = \frac{E_{nr} \cos \phi + IR}{E_{ns}} \quad (9)$$

$$E_{ndl} = \sqrt{I^2 R^2 + I^2 X^2} = IZ \quad (10)$$

$$X = 2\pi fL - \frac{1}{2\pi fC} \quad (11)$$

$$I = 1100 \sqrt{\frac{d^2 T}{R}} \text{ (for copper)} \quad (15)$$

$$D = \sqrt[3]{ABC} \text{ (see footnote, Table 93)} \quad (16)$$

Electrical transmission involving very high voltage, great distance, or interconnection of a number of large generating stations presents complicated electrical problems which are beyond the scope of this chapter. They have been discussed briefly in the preceding pages. The electrical problems of transmission properly occupy the field of the electrical engineer; he should be familiar as well with the principal structural and economic features which must be considered in determining the size of the conductor. In a like manner the civil engineer should understand the basic electrical principles necessary to transmission-line design. It is intended to outline for him here the method of computing the electrical factors which are of most importance: line loss, regulation, and maximum current capacity for three-phase three-wire 60-cycle transmission for not

over 66,000-volt 40-mile transmission, or 44,000-volt 60-mile transmission and to discuss in a general way the problems of electrical transmission.

Line Loss and Ohm's Law.—Line loss is the power expended in transmission. It depends fundamentally on the resistance of the conductor to the flow of electricity. This resistance is overcome by a drop in voltage or loss in electrical pressure, which drop for a given conductor is proportional to the flow of electricity through it. Ohm's law expresses this reaction as follows:

$$E_d = IR \quad (1)$$

which shows that the voltage drop or difference in potential due to resistance is equal in any conductor to the product of the current in amperes and the resistance in ohms. As the power expended is the product of the drop in voltage due to resistance and of the current in amperes, the line loss per conductor may always be expressed as

$$W_l = E_d I \quad (2)$$

or

$$IR \times I = I^2 R \quad (3)$$

This formula neglects the effect of inductance and capacity. It is, however, rigidly correct if the voltage drop used is that component in phase with the line current. Additional power may be lost through leakage, corona, and skin effect, all of which will be considered later.

It follows, then, that, if the current flowing in a conductor of known resistance can be determined, the line loss can be computed. Inductance and capacity are both present in alternating-current circuits, and each has the effect of disturbing the phase relation of the current and voltage. Hence, it is advisable first to describe the action of alternating current and to discuss the nature of inductance and capacity and their effect on the alternating-current circuit.

Nature of Alternating Current.—In the generation of alternating current a conductor is forced across a magnetic field, and an electromotive force or voltage is induced in the conductor which varies as the sine of the angle between the path of the magnetic flux and the plane of the conductor. The cycle is a complete revolution of the conductor through 360 electrical degrees, and the electromotive force induced varies as a sine curve, having equal maximum positive and negative values, and becomes zero at the beginning, middle, and end of the period.

In Fig. 246 the sine curve *ABCDE* shows a single complete cycle, the ordinate to which measured from the horizontal axis represents at any point or time the instantaneous value of the induced voltage in the conductor. The mean effective value of the voltage and current for the period is generally used instead of the maximum and is equal to the maximum or instantaneous value divided by $\sqrt{2}$.

The number of cycles or complete alternations per second is called the frequency. A frequency of 60 cycles has been generally adopted on account of the saving possible over that with 25 cycles in the cost of generators, motors, and transformers and also because of the steadier lighting effect obtainable with the higher frequency. To offset this advantage of lower cost of generation and energy conversion is the higher cost of transmission, an example of which was given in the preceding pages.

The current flowing obeys the same law as the voltage, as with a constant resistance it will be directly proportional to the voltage induced, and, when inductance and capacity are not present or are in a balanced condition, the value of the current at any time will be directly proportional to the induced voltage or, in electrical terms, the current and

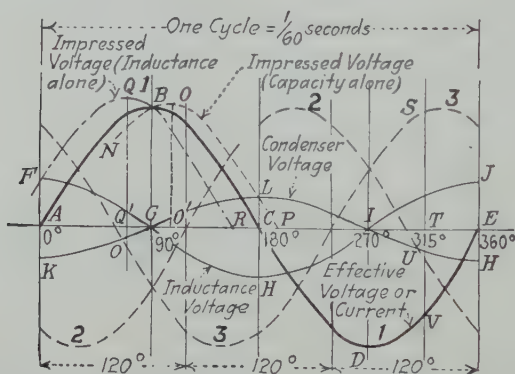


FIG. 246.—Instantaneous values of voltage and current in an alternating-current circuit illustrating the effect of inductance and capacity.

voltage will be in phase. Referring again to Fig. 246, the same curve *ABCDE* will represent the current variation under these conditions.

Inductance.—Inductance in a conductor or an alternating-current circuit is an induced voltage produced by the rapid changing or alternation of the current and is proportional to the rapidity of the current change. This induced voltage is due to the magnetic field set up by the current flow. As the current changes, the number of lines of force in the magnetic field changes, which induces this voltage in a way similar to the generation of electromotive force in a coil revolving through a magnetic field.

As shown by the *FGHIJ* in Fig. 246, this voltage or inductance is zero at 90 and 270 deg. when the current is at its maximum positive and negative values, as at this time the current change passes through zero and from an increasing to a decreasing quantity. Inductance is a maximum when the current is going through zero, as at this time the current change is most rapid as it approaches equality with the angular change in accordance with the sine relation.

Capacity.—The capacity of a conductor or any alternating-current circuit is its ability to store current. As the current is stored, it produces a proportional rise in voltage which opposes or is counter to the voltage impressed on the circuit shown by the curve *NBOLP* in Fig. 246, to be discussed later. Curve *KGLIM* of Fig. 246, also a sine curve, indicates both this counter or condenser voltage and the charging current or the current stored during a cycle. Starting at *G*, or 90 deg., the difference in potential between the two voltages is a maximum, as indicated by the curves, and the current is stored at the maximum rate as it passes through zero in accordance with the sine relation and continues to be stored at a rate proportional to the difference in voltage until the circuit is completely charged at *L*, or 180 deg., where the potential between the impressed voltage and the counter or condenser voltage equals zero. During the following 90 deg. the conductor discharges the stored current, and the condenser voltage drops to zero. The remaining portion of the cycle is the same as described for the first 180 deg., except for being opposite in sign.

The impressed voltage as shown through a part of a cycle by the curve *NBOLP* of Fig. 246 is the resultant of the two voltages, the effective voltage *ABCDE* and, considering capacity only, the condenser voltage *KGLIM*. This curve of impressed voltage is the algebraic sum of these two voltages and represents at any instant the magnitude and time of the voltage which must be delivered to counteract the condenser voltage *KGLIM* and leave the effective voltage in phase or in step with the current, as shown by the curve *ABCDE*. In a similar way, considering only the voltage of inductance *FGHIJ*, the impressed voltage necessary to counteract the inductance to produce the effective voltage *ABCDE* would be the algebraic sum of these curves or *FQBR*, also shown only through a part of a cycle.

Inductance and Capacity—Reactance.—To conclude with the description of the effects of inductance and capacity, it is evident from the diagram that inductance voltage lags exactly 90 deg. in time, as it passes through zero or reaches a maximum exactly 90 deg. behind the effective voltage, causing the impressed voltage to lag behind the effective voltage by the angular amount *RC* which scales 22 deg. from the figure. Capacity or condenser voltage leads exactly 90 deg. as it passes through zero and reaches its maximum 90 deg. ahead of the effective voltage and current, and in a similar way the impressed voltage, considering inductance only, leads the resultant or effective voltage by the angular amount *CP* which scales 15 deg. In each case the impressed voltage has a maximum value *OO'* and *QQ'* for capacity and inductance, respectively, which are in excess of the amount *BG* required for the effective voltage.

Figure 246 shows, as previously stated, the instantaneous values of the current and voltage for a single cycle and is convenient to use in

describing the properties of alternating current, inductance, and capacity. The average effective values, discussed on page 555, are those used, and these quantities can be shown as vectors with the average effective amounts proportional to their length and with their angular relation determining their direction. Figure 247 shows such vector diagrams giving the same relationship of average effective voltage, inductance, and capacity as in Fig. 246.

Figure 247(1) shows inductance causing the impressed voltage to lag 22 deg. which corresponds to the angular period $Q'G$ and RC in Fig. 246, and the ratio of the effective voltage to the impressed voltage is

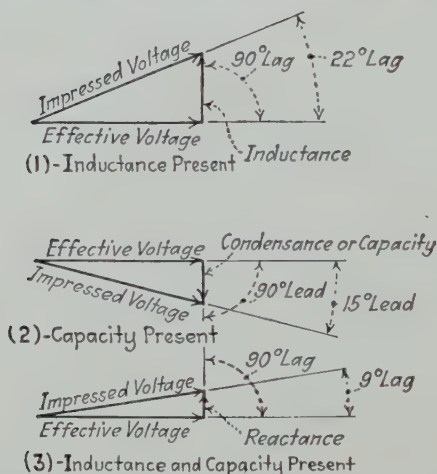


FIG. 247.—Average values of voltage and current in an alternating-current circuit. Illustrating the effects of inductance and capacity.

0.93, which is the same as the ratio BG/QQ' in Fig. 246 and is called the power factor, to be discussed in more detail later. In a similar way capacity, in Fig. 247(2), causes the impressed voltage to lead by 15 deg., corresponding to CP and GO' of Fig. 246, and the ratio of the effective and impressed voltage or power factor is 0.97, to compare with BG/OO' of Fig. 246.

The term reactance is given to the net effect of inductance and capacity. As they are always 180 deg. apart, they exactly oppose each other and tend to reduce the reactance. Another point of interest is that the effect of reactance on the power is zero, as it neither increases nor decreases the effective voltage or current, being at 90 deg. to both.

Figure 247(3) shows the usual case where inductance and capacity are both present. Here the effect of inductance is greater than capacity, as it is in the cases we are considering, and the reactance causes an impressed voltage lagging 9 deg. The corresponding curve in Fig. 246 has not been constructed, but it can readily be seen that the effect of

adding algebraically the instantaneous values of the curves of effective capacity and inductance voltage will result in such a lagging curve, as the inductance is numerically greater than the capacity.

Three-phase Three-conductor Circuit Relations.—So far we have considered only one phase. A three-phase three-wire circuit may be considered as three electrical circuits, each 120 deg. apart, as shown in Fig. 248 by the three vectors E_n — 01, 02, 03, which represent the magnitude and direction of the average effective voltage and current of each circuit considered separately, inductance and capacity being neglected for the sake of clearness. Fig. 246 shows the instantaneous values of

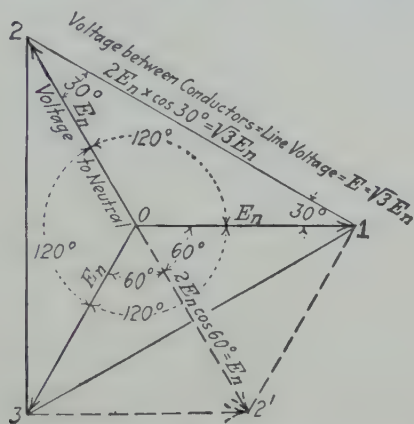


FIG. 248.—Phase relations—three-phase three-wire circuit.

the three phases by the sine curves 1-1, 2-2, and 3-3 which are shown at the interval of 120 deg.

The vector sum is zero, as the resultant of any two is exactly equal and opposite to the third so that the current between wires is zero at all times. The return wires for the separate phases can, therefore, be omitted and only three conductors used, as each acts alternately as a return. Figure 248 proves this relation by showing that the resultant 0-2' of currents 0-1 and 0-3 is opposite and equal to 0-2. Figure 246 also shows this to be true at any instant by the equality of the sine curves at any point chosen, as T at 315 deg. in the cycle, where $ST + TU + TV = 0$. Expressed numerically, $\sin 75 \text{ deg. } (0.966) + \sin 195 \text{ deg. } (-0.259) + \sin 315 \text{ deg. } (-0.707) = 0$.

The voltage E_n is, therefore, the voltage in each of the three phases and is called the voltage to neutral. The line voltage E or potential difference between the phases or conductors is the vector sum of any two of the voltages to neutral as represented by the line 1-2, which Fig. 248 shows to be equal to the $\sqrt{3} E_n$.

Power Factor.—Assume now, referring to Fig. 249, that a transmission circuit serves a load which can be considered simply as an extension of a

separate portion of this circuit and that this load also has inductance and capacity, resulting in an impressed voltage lagging by 36 deg. 52 min. The ratio of the effective voltage to the impressed voltage is in this case 0.80, or the cosine of the angle given above. This ratio or cosine of the angle of lag or lead or phase displacement ϕ is called, as stated before, the power factor, and in this case it is the power factor of the load. This figure divided into the actual power gives the apparent power to be provided to counteract the reactance of the load. The kilowatt-load demand at 0.80 power factor is $0.80 \sqrt{3} E_r I$, while the kilovolt-ampere or apparent-load demand is $\sqrt{3} E_r I$ or 25 per cent additional. While no more power is required on account of a low power factor, except indirectly,

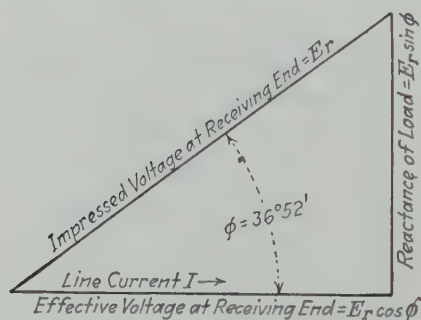


FIG. 249.—Vector diagram of load with 0.8 power factor.

$$\text{Load demand, kw.} = \frac{\sqrt{3} E_r \cos \phi I}{1000}$$

$$\text{Apparent load demand, kva.} = \frac{\sqrt{3} E_r I}{1000}$$

on account of increased losses, the conductors and machines have to be made sufficiently large to carry this apparent power, and this increase in size is inversely proportional to the power factor, so that it is of great importance to limit the reactance of the load as it adversely affects the power factor. The power factor of 0.80 is a common one for estimating purposes and is used here for illustration. As the power factor of the load is generally lagging, the effect of capacity in the form of synchronous motors has a corrective effect, while induction motors, particularly single phase, taking a lagging current for their magnetic circuit are the chief cause of a low power factor.

This chapter is concerned only with the transmission line, so that the transformers and other terminal apparatus in the circuit at both ends are not considered, nor is it usual to consider them in transmission-line computations of the scope to which this chapter is limited, hence their effect in loss and reactance should be added to the load at one end and subtracted from the output of the generator at the other end.

Line Loss, Line Drop, Regulation, Reactance, and Impedance Formulas.—The foregoing has been limited to a general discussion of the effects of inductance and capacity on any alternating-current circuit, and the character of a three-phase three-conductor circuit. Enough has been said to show that reactance, the effect of inductance and capacity, will be diminished by the presence of capacity. As capacity is also negligible at or at less than 66,000-volt 40-mile transmission, in comparison with other variables usually considered constant, it is, therefore, on the safe side to omit its effects.

Knowing the relation of voltage between lines and the voltage to neutral to be

$$E = \sqrt{3} E_n \quad (4)$$

and that each phase can be considered as a circuit by itself or that the power may be expressed as

$$W = 3E_n I \cos \phi \quad (5)$$

for the three conductors or circuits, the expression for the current per phase may thus be derived:

$$I = \frac{W}{\sqrt{3}E \cos \phi} \quad (6)$$

As the power, power factor, and voltage of the load are known, the line current per phase can then be found and the total losses determined by Eq. (3),

$$W_l = 3I^2R$$

Referring to Fig. 250, the graphical vector construction indicated shows one phase or considers the voltage to neutral and suggests a convenient analytical method of solution. The use of the voltage to neutral involves less chance for error, as one phase only is considered. The graphical method is awkward, as the loss in voltage is only 5 to 10 per cent of the line voltage and a large scale is necessary for accuracy, so that the analytical solution which follows is more often used.

The load-power triangle ABC shown involves one-third the load, the line-loss triangle CDE involves one-third the total loss, and the generated-power triangle $AB'E$ is the vector sum of the two and represents one-third the power generated, each phase being considered as a separate circuit taking one-third the power, with the voltage to neutral being used throughout.

E_{ns} , the voltage to neutral at the sending end, then equals the vector sum of AB' and $B'E$, or $\sqrt{(AB')^2 + (B'E)^2}$. Replacing these terms with the known quantities, we have

$$E_{ns} = \sqrt{(E_{nr} \cos \phi + IR)^2 + (E_{nr} \sin \phi + IX)^2} \quad (7)$$

From Table 88, $R = 0.07935 \times \frac{5280}{1000} = 0.418$ ohm per mile

or $\frac{0.418}{0.97} = 0.432$, correcting for hard-drawn copper.

From Table 92, $X = 2\pi fL = \frac{2 \times 3.14 \times 60 \times 0.418}{1000}$ (11)
 $= 0.158$ ohm per 1000 ft.
 $= 0.830$ ohm per mile.

Hence, for 44 miles:

and $R = 0.432 \times 44 = 19.0$ ohms
 $X = 0.830 \times 44 = 36.6$ ohms
 $IR = 109 \times 19.0 = 2070$
 $IX = 109 \times 36.6 = 3990$

By Eq. (7) $E_{ns} = \sqrt{(38,000 \times 0.8 + 2070)^2 + (38,000 \times 0.6 + 3990)^2}$
 $= 42,100$ voltage to neutral

and $E_s = 42,100 \times \sqrt{3} = 72,800$ volts.

$\cos \phi' = \frac{(38,000 \times 0.8 + 2070)}{42,100} = 0.77$ power factor of line at sending end.

By Eq. (8) $\frac{72,800 - 66,000}{66,000} = 10.3$ per cent regulation.

By Eq. (10) $\sqrt{(109 \times 19.0)^2 + (109 \times 36.6)^2} = 4490$ volts, line drop to neutral,
and line drop $= 4490\sqrt{3} = 7780$ volts.

By Eq. (3) $\frac{3 \times 109^2 \times 19.0}{1000} = 678$ kw. line loss,

and line loss $= \frac{678}{10,678} = 6.35$ per cent.

Electrical and Physical Properties of Stranded Wires.—In addition to copper and aluminum, which are commonly used for conductors, steel and iron are also required for ground, guy, and telephone wires, and steel is used as a conductor in combination with copper and aluminum for long spans and occasionally alone for either long special spans or for transmission of a small quantity of power. A conductor composed of a central core of one or more steel wires surrounded by one to three layers of aluminum wires called "aluminum cable-steel reinforced" or A.C.S.R. is quite commonly used for high-tension transmission, where a large diameter is desirable on account of corona and high strength is necessary due to the long spans used. A "copper-clad" or "copper-weld" strand is sometimes used for special long-span construction, each wire of the strand consisting of a core of steel and external portion of copper. As this type of wire is not in general use for standard line transmission, its characteristics are not given.

Table 86 (see page 578) gives the physical and electrical properties of these materials for stranded wires, with the exception of the iron wires, which are solid and small in size, although they can be obtained stranded. The steel and iron wires are always furnished galvanized, and for usual transmission purposes all wires are furnished bare, as the insulation or weatherproofing sometimes used in distribution systems or low-voltage transmission through cities does not afford protection unless the voltage is

low, and it also adds materially to the weight of the wires, increases the snow and ice loading, and is not durable. A considerable variation will be noted in the table in the strength, elastic limit, and modulus of elasticity, not only in the different materials but also in the different commercial grades of the materials.

The figures given in Table 86 are not recommended for specific use where there is a considerable variation, as the manufacturers will supply the exact data for the particular wire in question, but the figure underlined or the average of the two extremes can be used for comparative and preliminary purposes.

This variation is due to the stranding of the wire and the size of wires used in the strand. Stranding causes a reduction in the unit strength and elastic modulus when compared to that of the separate conductors of which it is composed; 90 per cent is a fair average figure generally used for this decrease due to stranding, but it varies depending on the size and number of wires and the method of making up the strand. All wires after being rolled hot to a given diameter are drawn cold successively through smaller and smaller tapered holes, the hardness being proportional, and the ductility inversely proportional, to the amount of this work done. After the wire becomes so hard that it cannot be drawn further, it is annealed or heated, which restores the ductility and reduces the stress, and the drawing can then be continued. The hardness affects the surface more than the interior of the wire, which causes an increase in strength for small wires, which are also stronger and less ductile than the larger sizes due to the greater number of drawings necessary.

This variation in strength with size of wire affects the strength of the strand, but in no regular way, as a strand of equal-diameter wires can be made up of few or many wires, the strength usually increasing with the number used. The number of wires that will strand are 3 and 7, 19, 37, 61, 91 or more wires built up in one, two, three, four, five, or more layers about a single central wire, with the diameter of the whole equal to the diameter of the single wire multiplied by twice the number of layers plus one, as illustrated in Fig. 251 for a 19-wire two-layer strand.

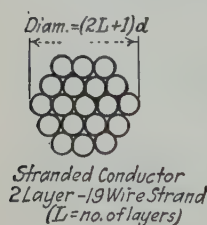


FIG. 251.

There is, therefore, a definite advantage in the use of a stranded wire if the increase in strength of the individual wires over a single solid wire makes up for the loss in strength due to stranding. The principal reason, however, for using stranded instead of solid wires is the stiffness of large solid wires, which the tables will show are limited to about $\frac{1}{2}$ -in. size. When used with pin insulators, the wire is held quite rigidly in place, while the wire between supports is always moving in the wind, resulting in a constantly reversing bending moment at the support,

which causes failure by crystallization or fatigue of the metal in a solid conductor much more quickly than in the more flexible stranded conductor of equivalent cross section. Another reason for using stranded wire is due to the greater safety factor of a strand in comparison with a solid conductor. A nick in the surface of a hard-drawn conductor cutting through the hard and strong skin may seriously weaken a solid conductor, where with a stranded conductor the damage would not be so serious, as it would usually be confined to a single wire in the strand.

Table 86 also shows the coefficient of resistance for the different materials per degree from 20° on the centigrade scale. This increase or decrease in resistance is computed in the same manner as the increase in length due to expansion except for the difference in the temperature scale.

There are many gages for wires, only two of which are of interest in transmission: the Brown & Sharpe or American wire gage, which is standard for all conductors except iron and steel, for which the steel wire gage or American Steel and Wire Company's gage is standard. Electrical conductors are also listed by the area of the conducting material in circular mils, which is the square for single wire, and the sum of the squares for a strand or cable of the diameter of the wire in thousandths of an inch, which when multiplied by $\pi/4$ and divided by 1,000,000 equals the actual cross-sectional area of the conductor in square inches. As the B. & S. gage governs conductors, the area in circular mils is thus the square of that diameter of the wire in thousandths of an inch, as given in Table 87 (as well as the area in square inches), which also shows the diameters of single wires corresponding to the number of the wire gage for both gages mentioned.

Tables 88 to 93 inclusive give the mechanical and electrical data necessary for the purpose of this chapter for stranded hard-drawn copper, stranded aluminum, and A.C.S.R., the usual transmission-line conductors. (Tables 86 to 93 inclusive are given on pages 578-589.) Tables for steel, iron, and other grades of copper and special strands such as copper weld can be found in electrical handbooks and manufacturers' catalogues. Tables 89 and 90 for aluminum and A.C.S.R. conductors use the words "copper equivalent," which means the size of copper wire of the same conductivity. Table 91 shows the variation of alternating-current resistance with current density. The figures given include the resistance due to skin effect, as well as the increased resistance due to higher temperatures of the conductor with increased current.

Tables 92 and 93 give values of self-induction for stranded copper and A.C.S.R. for equilateral spacing of the conductors, and Table 93 gives, as does Table 91, the variation for different current densities. A footnote for Table 93 shows a method of computing the equivalent equilateral spacing for any other position of the conductors. These tables

give self-induction in mil-henrys per 1000 ft. and per mile of conductor. The reactance value X in ohms, including inductance and capacity, is

$$X = 2\pi fL - \frac{1}{2\pi fC} \quad (11)$$

Neglecting capacity, the second term in the equation, and assuming 60-cycle current, the factor $2\pi f$ becomes $6.28 \times 60 = 377$. If, then, the values in the tables are multiplied by 377 and divided by 1000 to reduce mil-henrys to henrys, the reactance in ohms per unit length of conductor will be obtained.

The table for self-induction for all aluminum conductors is not included. It can be approximately figured, however, from the following relation given by the manufacturers:¹ "If the distance between the wires in an aluminum circuit is 25 per cent greater than in a copper circuit, the self-induction will be the same in the two cases." An examination of Table 93 shows that this difference is small and usually negligible for ordinary spacing.

Skin Effect, Corona, and Leakage.—Skin effect is another secondary result of alternating current similar to inductance and capacity and is caused wholly by the rapid alternation, which crowds the current to the outer portions of the wire. Its current density, therefore, is not constant, and the voltage drop will be greater than that induced by the same amount of direct current in the same wire. To allow for this greater drop, the resistance is increased in proportion.

Table 85 shows the coefficients to be applied to the resistance of copper and aluminum wires to allow for skin effect. It will be noted that skin effect increases with the frequency and with the size of the wire, and further study will show that it is the same for aluminum and copper of equal conductivity and that with 60-cycle current it may be neglected in ordinary computations where the conductor size is not more than 500,000 cir. mils, which is larger than the usual conductor for the voltages being considered. For A.C.S.R., as stated before, the added resistance due to skin effect is included in the resistance as given in Table 91.

For steel and copper-clad steel wires there is an appreciable loss due to skin effect, but, as neither are used ordinarily for line conductors, they do not come within the limits of this chapter.

Corona, described previously, is an interesting phenomenon which starts at a voltage which increases directly with size, spacing, and smoothness of the conductor and decreases with altitude, temperature, and frequency, but the voltages considered are lower than that at which this loss of power will take place unless the size of wire is much smaller than common for structural reasons, for the voltage and spans considered, or the elevation much greater than sea level. For example, a 1/0 conductor,

¹ "Aluminum Electrical Conductors," Aluminum Company of America, 1920.

TABLE 85.—SKIN-EFFECT FACTORS AT 20° C. FOR COPPER AND ALUMINUM WIRES
(By courtesy of the National Electric Light Association, "Handbook on Overhead Line
Construction," 1914)
(For straight wires having circular cross section)

Product of circular mils by cycles per second	Factor for	
	Copper wire, $C = 100$	Aluminum wire, $C = 62$
5,000,000	1.000	1.000
10,000,000	1.000	1.000
20,000,000	1.008	1.000
30,000,000	1.025	1.006
40,000,000	1.045	1.015
50,000,000	1.070	1.026
60,000,000	1.096	1.040
70,000,000	1.126	1.053
80,000,000	1.158	1.069
90,000,000	1.195	1.085
100,000,000	1.230	1.104
125,000,000	1.332	1.151

NOTE: There is a difference of opinion concerning the factors controlling skin effect in copper and aluminum wires. These tables should be used cautiously especially for cables larger than 1 mill. cir. mils.

which in the northeastern section of the United States is about the smallest advisable for steel-tower line work, where the spans are of 500 ft. and over, on a corresponding minimum allowable spacing of 7 ft. would have to transmit 92,000 volts before corona would start at sea level. At 9000 ft. or at barometric pressure corresponding to this height corona would occur with 66,000-volt transmission, as the starting voltage of corona is reduced in direct proportion to the barometric pressure.

Corona will, therefore, not be often met with below 66,000 volts except in special cases. In such cases the losses in power would be proportional to the square of the voltage in excess of that at which corona begins. These losses will usually be so great as to require a larger conductor which under ordinary weather conditions will be free from corona, and, therefore, this loss, except during severe storms, is very rarely encountered.

Leakage, which is the actual loss in potential across the insulators due to rain, mist, snow, dirt, or dust, may also be neglected on such short, low-voltage lines as are being considered here.

Peek¹ has derived empirical formulas for disruptive critical voltage of corona, E_0 in kilovolts and power losses P in kilowatts per mile, as follows:

¹ PEEK, F. W., JR., "Dielectric Phenomena," McGraw-Hill Book Company, Inc., New York.

$$E_0 = 213m_0rd \log_{10} \frac{D}{r} \quad (12)$$

$$P = \frac{0.00390}{d}(f + 25)\sqrt{\frac{r}{D}}(E - E_0)^2 \quad (13)$$

where d = relative air density = $\frac{17.9b}{459 + t} = 1$ under standard conditions

when $b = 30$ in. and $t = 77^\circ\text{F}$.

b = barometric pressure, inches of mercury.

t = temperature, degrees Fahrenheit.

f = frequency, cycles per second.

r = wire radius, inches. It is the radius of the circumscribing circle in a stranded cable.

D = interaxial spacing in inches for single-phase or equilateral three-phase lines.

e = effective line voltage, kilovolts.

m_0 is an irregularity factor:

1.00 for polished wires.

0.98 to 0.93 for roughened or weathered wires.

0.87 to 0.83 for seven-strand cables, concentric lay.

0.85 to 0.80 for 19-, 37-, and 61-strand cables, concentric lay.

This factor tends to improve as the conductor weathers and becomes smoother. A value of 0.85 is safe to use for large concentric-lay cables.

Formula (12) is applicable to good weather conditions. Storms, particularly of snow, considerably increase corona losses. This condition is taken account of in Eq. (12) by using 0.80 E_0 in place of E_0 . This formula for power loss holds accurately above a critical voltage E_v , called visual critical voltage, at which corona appears all along the line. E_v is higher than E_0 and its value is given by the formula

$$E_v = 213m_vdr \left(1 + \frac{0.189}{\sqrt{dr}} \right) \log_{10} \frac{D}{r} \quad (14)$$

where m_v , the irregularity factor,

= 1.00 for polished wire.

= 0.72 for local corona all along the cable.

= 0.82 for decided corona all along the cable.

Maximum Current Capacity.—On short, heavily loaded lines the size of conductor determined by regulation or power loss may be so small that the current density will be high enough to cause the temperature in the conductor to rise beyond a safe limit, in which case a larger conductor will be required. An empirical formula for solving for the allowable current for bare stranded overhead wires out of doors for a given temperature rise from 25°C . is as follows:

$$I = 1100 \sqrt{\frac{d^3 T}{R}} \quad (15)$$

where d is the diameter of the wire in inches, T is the temperature rise above 25°C., and R is the resistance of the conductor in ohms per mil-foot at a temperature of $T + 25^\circ\text{C}$. At 100°C. (212°F.) hard-drawn copper begins to anneal and for that reason a 40°C. or 72°F. rise which will limit the temperature to 65°C. or 149°F. with air at 25°C. or 77°F. is adopted as a safe maximum figure by Taylor and Neale, in their book, "Electrical Design of Overhead Power Transmission Lines." R for this case for copper of 100 per cent conductivity is 12.2 and 19.0 for aluminum, and the values may be then simplified to read $I = 2000d^{3/2}$ for copper and $1600d^{3/2}$ for aluminum.

Transposition.—Transposition is the changing of the relative position of conductors in a circuit to counteract the effect of either self-induction or mutual induction. Considering first the effects of self-induction, which is induction within a circuit, there will be no need of transposition if the three conductors are located at the apices of an equilateral triangle, as each wire will carry an equal share of the inductance. If, however, the wires lie in a plane, the middle conductor will be more affected than the outer ones. If each conductor occupies in order the position left, middle, right, left, etc., or top, middle, bottom, top, etc., then the inductance effect will be equalized, and the average separation as given in Tables 92 and 93 may be calculated in accordance with the formula given in the footnote of Table 93.

Mutual inductance, or the inductance between two circuits, may be neglected if both circuits lie in a single plane. If they lie in two planes, as is the more common case, the transposition should be opposite in the two circuits, as top, middle, bottom, top, etc., for one circuit and bottom, middle, top, bottom, etc., for the second circuit. It is the general practice to avoid transposition in transmission circuits as far as possible to simplify construction, as special towers are required for the purpose. Where the opportunity offers, transpositions are commonly made where special towers required for other purposes, such as switching or spreading towers at crossovers or special long-span structures, allow the change to be made. In any case the necessity for transposition of circuits depends on the extent to which the phase relations are disturbed and is much more of a problem in high-voltage transmission, where for 220,000 volts special transposition towers are necessary at regular intervals.

Insulators and Hardware.—Insulators are a very important part of a transmission line as the character of service rendered depends very largely on their reliability. The theory and design of insulators are really the problem of the manufacturers, who have developed standard insulators and hardware for the different types and voltages required, and these

standards should be used where possible. The following will be limited to a description of the various types and the more important facts concerning their use.

The purpose of the insulator is to insulate the conductor or prevent short-circuiting and leakage. The use of a non-conductor or dielectric is necessary, porcelain being used almost universally for the purpose. The size and shape of the insulator are determined by the voltage which it must resist. This resistance is proportional to the radial or leakage distance measured on a diametrical cross section along the surfaces of the insulator from the conductor to the ground, such as the line *AB* in Fig. 252.

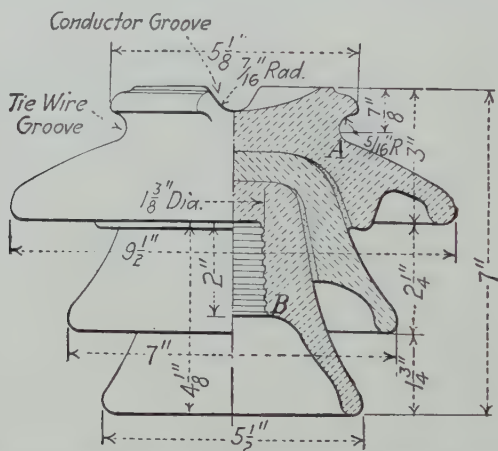


FIG. 252.—Standard pin-type insulator. Approximate characteristics. (By permission of R. Thomas & Sons Co.)

Nominal rating.....	35,000 volts	Leakage distance.....	19 in.
Dry flash-over.....	120,000 volts	Wet arcing distance.....	$6\frac{1}{4}$ in.
Wet flash-over.....	80,000 volts	Minimum pin height.....	$7\frac{1}{4}$ in.
Net weight 13 lb.			

When the voltage is reached at which the resistance of the insulator fails, any one of three things may happen: first, the current may "flash over" or arc from the wire to the nearest conducting material; second, it may "cascade" or successively arc across and flow over the surfaces of the insulator; third, it may puncture the insulator. As the last is an actual failure, it is a governing principle of design that the insulator should have a greater factor of safety against puncture than against flashover, the ratio of the safety factors being 1.35 or more.

Lightning, short circuits, and voltages of high frequency due to disturbances on the line are the cause of flashovers. A lower flashover voltage results from lower barometric pressure and from accumulations of dust, dirt, or sleet on the surface of the insulators.

When high-frequency potentials cascade from the conductor across the insulator to ground, due to lightning or surges on the line, frequent

failures of insulators may occur. As a protection against this, the arcing horn for the lower and the ring or shield for the highest voltages have come quite generally into use, resulting, however, in lower flashover voltage. The type of arcing horn used by the Turners Falls Company is shown in Fig. 253, attached to a string of six suspension insulator units. As the figure indicates, the arcing horns are so shaped that the arc will occur between their points, rather than cascade across the insulator units. The ring or shield is illustrated in Fig. 254. Here the arc is distributed around the circumference of a metal ring instead of a single point, thus reducing by diffusion the liability of insulator failure by puncture during flashover.

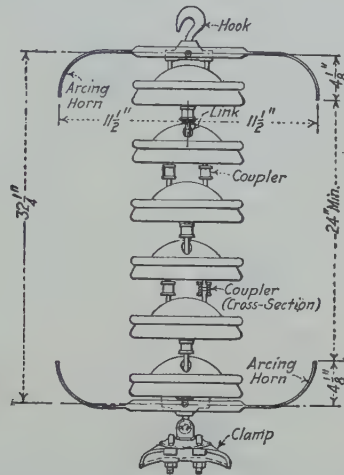


FIG. 253.—Six unit 10-in. Hewlett suspension insulator string and hardware for 110,000 volts—Turners Falls Power Company.

Insulators may be divided into two types, pin and suspension, differing in that the units of the pin insulator are rigidly fastened together, while some movement is possible between the units of the suspension insulator. The pin-type insulator is also supported rigidly by a vertical steel pin bolted to the cross arm while the string of suspension insulators or units is hung from the cross arm by a connection which allows lateral movement.

The pull of the wire passing over the top of the pin insulator produces a bending stress in the insulator as well as in the pin, while in the suspension insulator the stress is wholly axial, as the flexible connections eliminate the bending moment. The bending stress in the wire when rigidly held at the supports has been discussed and is another troublesome feature of pin-type insulators. The desirability of eliminating these bending stresses has led to the adoption of the flexible suspension insulator, which has made it practical and economical to increase greatly the arcing

distance to allow the use of much higher voltages than ever will be feasible with the pin-type insulator, which now can be built for about 60,000 volts but is little used above 44,000 which is the usual limiting practical voltage. Another advantage of the suspension type of insulator lies in the fact that it is better protected from lightning, as the conductor is below, not above the nearest ground, as with the pin insulator.

Figures 252 and 255 show fully a standard pin-type insulator and metal "Lee pin" for use with this particular insulator. This insulator

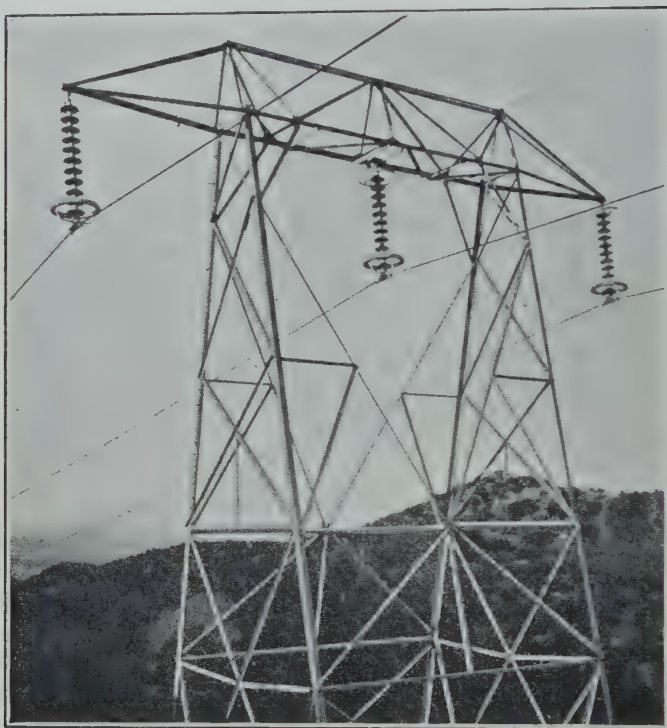


FIG. 254.—Standard single-circuit suspension tower, 220,000 volts—Southern California Edison Company. (Courtesy of Aluminum Company of America.)

is a standard three-part cemented porcelain insulator designed for 35,000-volt transmission.

The conductor passes through the central groove on the top of the insulator and is held in place by annealed tie wires of the same material which are wrapped with usually not less than six turns around the conductor, then passed completely around the tie-wire groove, and fastened with the same number of turns to the conductor on the other side of the insulator. For No. 4 and No. 6 line wire No. 6 wire is usually specified, and No. 4 for 1/0 to No. 2 and No. 2 for wires larger than 1/0.

The Lee pin shown in Fig. 255 consists of a thimble, which is usually cemented into the insulator, the base, which furnishes a wide bearing

on the cross arm, and a bolt, which completes the assembly when fastened to the cross arm. The pin and insulator are usually furnished by the same manufacturer, as any bending in the pin will crack the insulator.

Where aluminum is used on pin-type insulators, a flat wire wrapping of aluminum is placed around the conductor before the tie wire is attached. This flat wire tends to protect the aluminum at the point of support, as it distributes the wear and shock of vibration due to swinging of the conductor.

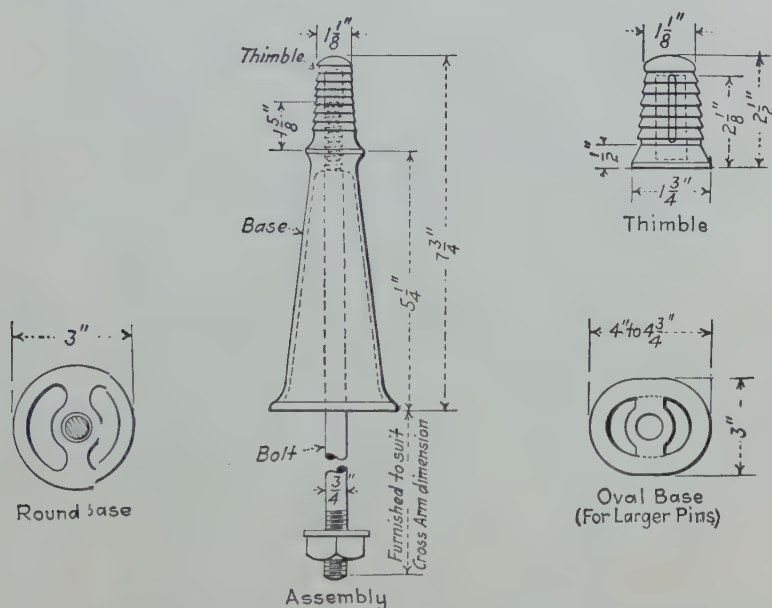


FIG. 25.—Assembly of all-metal "Lee pin" No. 9008, with separate thimble. Net weight 3.2 lb., for wood cross arms. Net weight 3.14 lb., for metal cross arms. (R. Thomas & Sons Co.)

Figure 256 shows the standard cemented type of suspension insulator and Fig. 257 the later Thomas link-type Hewlett suspension insulator with accompanying tables giving the weight and length and voltage for dry and wt flashovers with varying numbers of units in the string.

The Hewlett type of insulator was developed at the time when a good deal of trouble was being experienced with suspension insulators of the cemented type. This was largely due to the differential expansion and contraction between the cement and the porcelain and resulted in cracking or puncturing the insulator, causing an electrical failure. This difficulty has been overcome, and the cemented type is now reliable in this respect. The Hewlett type puts the porcelain in compression, is not quite so strong, allows greater flexibility, and in case of failure of the porcelain the links will support the conductor; and of most impor-

tance, the life of the Hewlett will exceed that of the cemented type on account of its comparative freedom from internal stresses due to its homogeneous nature.

Returning to the question of flashover voltage, an examination of the two tables will show very little difference in either type. It will be noted

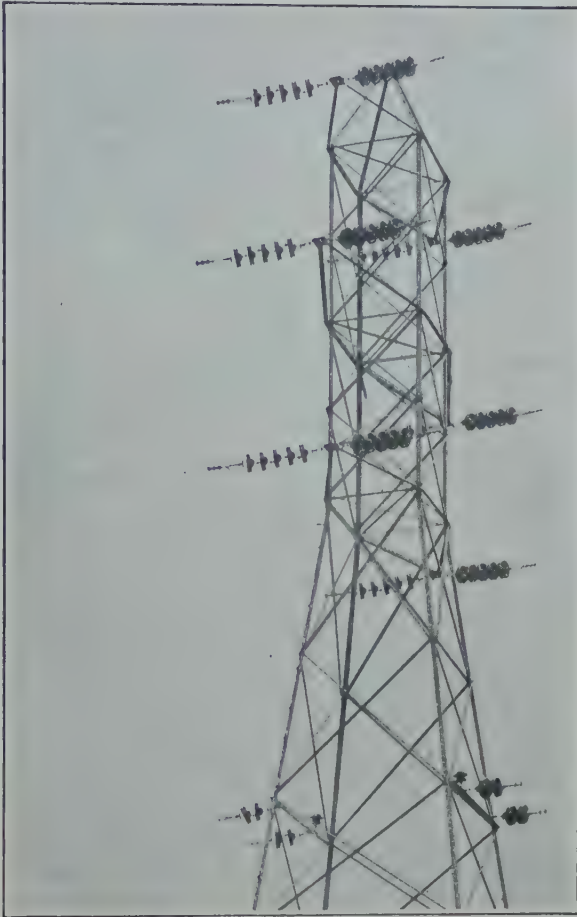


FIG. 258.—Standard angle tower, 66,000 volt line of Turners Falls Power and Electric Company, showing insulators in strain position and method of guying tower.

in Fig. 256 that, while the first unit is good for a dry flashover voltage of 85 kv., the second unit adds only 63 kv. and the third 56 kv., etc., which shows that the distribution of dry flashover voltage across the string is not uniform. The wet flashover voltage is very nearly proportional to the number of insulators, and the value of the wet flashover, therefore, becomes relatively larger with increasing voltages. For the suspension position an extra insulator is usually added to the number required to give a factor of safety of two against wet flashover. This results in a string of

four for 66,000 volts and six for 110,000 volts, as illustrated in Fig. 253, and 12 for 220,000 volts, although at the latter voltage 11 are shown in Fig. 254, the extra insulator in this case not giving so much additional margin of safety as with the lower voltage. For the strain position an additional unit is added to the string, as shown in Fig. 258, and, when the stress in the conductor is great, it is necessary to use a double or a triple string, as in Fig. 259, where a special conductor was used to give the required 65-ft. clearance above mean low water on several spans ranging

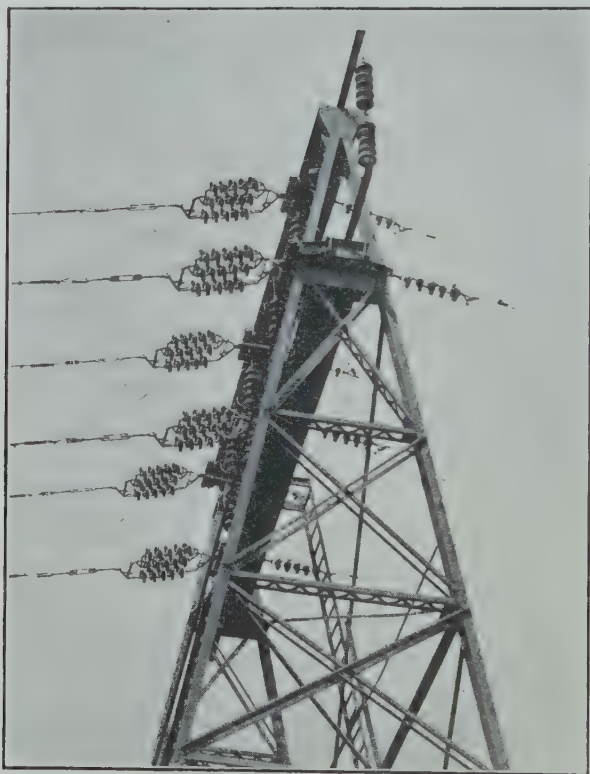


FIG. 259.—Dead-end tower, Turners Falls Power and Electric Company, showing insulators in strain position on long river crossing span.

from 1000 to 1500 ft. over the Connecticut River. The use of an equalizer bar is very clearly shown here, its purpose being to insure the delivery of an equal load to each of the three strings of insulators.

Figure 253 shows an assembly for 110,000-volt transmission of six 10-in. Hewlett insulators and complete hardware, including hook and clamps and the arcing horns already mentioned. Figure 257 shows the three customary means of attachment of the insulator to the cross arm, the hook, eye, and clevis adapter, which need no further discussion. The clamp as shown in Fig. 253 for the suspension type of insulator and in Fig.

258 for dead ends is important, as it must prevent the wire from bending on too sharp a radius and yet allow free motion, to minimize the effect of bending in the conductor. A copper sleeve or other protection for the conductor is always provided, and in the case of aluminum, as in the pin-type insulator, a flat aluminum wrapping wire is used to prevent damage to the conductor from the bolts which clamp the wire.

The clamp and, more especially, the tie wire are not so strong as the wire, which will usually slip before breaking, as shown in Fig. 275 by the string of insulators at the left suspended from the wire. This is an important point, as sometimes structures are designed for more load than can ever be exerted on them. One of the latest high-voltage¹ lines with very heavy conductors and loading is designed to take advantage of this fact by using a clamp designed to hold the conductor under all unbroken conditions but to slip and relieve the stress when the conductor is broken. Another feature of this line is the use of a ground-hinged, guyed steel pole or mast for each conductor at angles and dead-end points.

¹ "Wallenpaupack 220-kv. transmission line," *Elec. World*, July 24, 1926, p. 173.

TABLES GIVING MECHANICAL AND ELECTRICAL PROPERTIES OF TRANSMISSION-LINE CONDUCTORS
TABLE 86.—ELECTRICAL AND PHYSICAL PROPERTIES OF STRANDED CONDUCTORS

	Average alumi- num	Soft- drawn or annealed copper	Medium- drawn copper	Hard- drawn copper	E.B.B. ¹ iron (solid)	B.B. ¹ iron (solid)	Standard guy-wire steel	Siemens- Martin steel	Crucible or high- strength steel	Plow or extra- high- strength steel
Electrical properties										
Resistance per mil- feet at 20°C.....	17.0	10.37		10.57	62.9	74.8		119.7	122.5	125.0
Temperature coefficient of resistance per de- gree centigrade from original temperature of 20°C.....	0.0039	0.00393		0.00393	0.005	0.005		0.005	0.005	0.005
Melting point, degrees centigrade.....	625	1100	1100	1100	1635					
Physical properties										
Specific gravity.....	2.68-2.71	8.89	8.92	8.94	7.77	7.77		7.85	7.85	7.85
Elastic limit, 1000 lb. per square inch.....	14-16	6-15	20-30	30-35	25-30			35-55	70-80	105-115
Ultimate strength 1000 lb. per square inch...	23-27	30-40	40-50	50-68	50-55	60	57-70	60-118	125-150	180-190
Modulus of elasticity, million pounds per square inch.....	9-10	5-14	10-14	12-15	22-27			16-29	19-29	19-29
Coefficient of linear ex- pansion per degree Fahrenheit.....	0.0000128	0.0000095	0.0000095	0.0000095	0.0000067	0.0000067		0.0000067	0.0000067	0.0000067

¹ B.B. = Best Best, used primarily for telephone service; E.B.B. = Extra Best Best used primarily for telegraph service.

TABLE 87.—DIAMETERS OF WIRES IN DECIMALS OF AN INCH
(Cross-sectional areas in circular mils and square inches)

Number of wire gage	American Steel & Wire Co.'s steel wire gage	American wire gage (B. & S.)	Area in circular mils, B. & S. gage	Area in square inches, B. & S. gage
00000	0.4305	0.51650		
0000	0.3938	0.46000	211,600	0.166190
000	0.3625	0.40964	167,772	0.131770
00	0.3310	0.36480	133,079	0.104520
0	0.3065	0.32486	105,625	0.082958
1	0.2830	0.28930	83,694	0.065733
2	0.2625	0.25763	66,358	0.052117
3	0.2437	0.22942	52,624	0.041331
4	0.2253	0.20431	41,738	0.032781
5	0.2070	0.18194	33,088	0.025987
6	0.1920	0.16202	26,244	0.020612
7	0.1770	0.14428	20,822	0.016354
8	0.1620	0.12849	16,512	0.012969
9	0.1483	0.11443	13,087	0.010279
10	0.1350	0.10189	10,384	0.0081553
11	0.1205	0.09074	8,226.5	0.0064611
12	0.1055	0.08081	6,528.6	0.0051276
13	0.0915	0.07196	5,184.0	0.0040715
14	0.0800	0.06408	4,108.8	0.0032271
15	0.0720	0.05706	3,260.4	0.0025607
16	0.0625	0.05082	2,580.6	0.0020268
17	0.0540	0.04525	2,052.1	0.0016117
18	0.0475	0.04030	1,624.1	0.0012756
19	0.0410	0.03589	1,288.8	0.0010122
20	0.0348	0.03196	1,024.0	0.00080425

TABLE 88.—STRANDED HARD-DRAWN BARE COPPER CONDUCTORS
(By courtesy of the American Steel and Wire Company)

American wire or B. & S. gage	Circular mils	Number of wires in strand	Diameter each wire, inches	Diameter of strand, inches	Weight per 1000-ft. strand, pounds	Area strand, square inches	Resistance per 1000 ft. at 68°F. or 20°C.	Ultimate strength, pounds ¹
	1,000,000	61	0.1280	1.1520	3,081	0.78494	0.01060	45,800
	950,000	61	0.1248	1.1232	2,927	0.74618	0.01115	43,800
	900,000	61	0.1215	1.0935	2,773	0.70724	0.01179	41,700
	850,000	61	0.1181	1.0629	2,619	0.66852	0.01247	39,500
	800,000	61	0.1145	1.0305	2,465	0.62810	0.01325	37,300
	750,000	61	0.1109	0.9981	2,311	0.58922	0.01413	35,100
	700,000	61	0.1071	0.9639	2,157	0.54954	0.01514	32,900
	650,000	61	0.1032	0.9288	2,003	0.51020	0.01630	30,600
	600,000	61	0.0992	0.8928	1,849	0.47146	0.01767	28,400
	550,000	37	0.1219	0.8533	1,694	0.43181	0.01925	25,400
	500,000	37	0.1162	0.8134	1,540	0.39237	0.02116	23,200
	450,000	37	0.1103	0.7721	1,386	0.35234	0.02349	21,000
	400,000	37	0.1040	0.7280	1,232	0.31431	0.02648	18,900
	350,000	37	0.0973	0.6811	1,078	0.27512	0.03026	16,600
	300,000	19	0.1257	0.6285	923	0.23591	0.03531	13,800
	250,000	19	0.1147	0.5738	769	0.19635	0.04233	11,700
0000	211,600	19	0.1055	0.5275	647.1	0.16609	0.04997	9,980
000	167,772	19	0.094	0.4700	513.2	0.13187	0.06293	7,930
00	133,079	7	0.1378	0.4134	405.9	0.10429	0.07935	6,050
0	105,625	7	0.1228	0.3684	321.7	0.08303	0.10007	4,890
1	83,694	7	0.1093	0.3279	255.2	0.06559	0.12617	3,910
2	66,358	7	0.0973	0.2919	202.4	0.05205	0.15725	3,130
3	52,624	7	0.0867	0.2601	160.5	0.04132	0.19827	2,490
4	41,738	7	0.0772	0.2316	127.3	0.03276	0.25000	1,980
6	26,244	7	0.0612	0.1836	80.0	0.02059	0.39767	1,260

¹ Note that the ultimate strength given here is not a part of the original table but was calculated from the strength of the individual wires and multiplied by 0.90 to allow for stranding.

Resistance as given is for annealed copper which is about 3 per cent less than that of hard-drawn copper.

TABLE 89.—ALL-ALUMINUM CABLE—BARE
(By courtesy of the Aluminum Company of America)

B. & S. gage	Area		Copper equivalent C.M. or number	Usual stranding	Elastic limit, pounds	Ultimate strength, pounds	Ohms per 1000 ft. at 20° C.	Diam- eter, inches	Weight, pounds per 1000 ft.
	Circular mils	Square inches							
	1,590,000	1.249	1,000,000	61 × 0.1615	17,500	30,000	0.0109	1.454	1493
	1,515,000	1.190	950,000	61 × 0.1577	16,650	28,600	0.0114	1.419	1423
	1,431,000	1.124	900,000	61 × 0.1533	15,750	27,000	0.0121	1.380	1345
	1,351,500	1.061	850,000	61 × 0.1490	14,850	25,500	0.0127	1.341	1270
	1,272,000	0.9990	800,000	61 × 0.1445	14,000	24,000	0.0135	1.301	1195
	1,192,500	0.9366	750,000	61 × 0.1398	13,100	22,500	0.0145	1.257	1120
	1,113,000	0.8742	700,000	61 × 0.1351	12,250	21,000	0.0155	1.215	1046
	1,033,500	0.8117	650,000	37 × 0.1672	11,350	19,500	0.0167	1.170	971
	954,000	0.7493	600,000	37 × 0.1606	10,500	18,000	0.0181	1.124	896
	874,500	0.6868	550,000	37 × 0.1538	9,600	16,500	0.0197	1.077	822
	795,000	0.6244	500,000	37 × 0.1465	8,750	15,000	0.0217	1.026	747
	750,000	0.5890	472,000	37 × 0.1425	8,250	14,140	0.0230	0.994	705
	715,500	0.5620	450,000	37 × 0.1391	7,870	13,500	0.0241	0.974	672
	636,000	0.4995	400,000	37 × 0.1312	7,000	12,000	0.0271	0.918	598
	556,500	0.4371	350,000	19 × 0.1711	6,120	10,500	0.0311	0.856	523
	500,000	0.3927	314,500	19 × 0.1623	5,500	9,420	0.0347	0.810	469
	477,000	0.3746	300,000	19 × 0.1585	5,240	9,000	0.0363	0.793	448
	397,500	0.3122	250,000	19 × 0.1447	4,370	7,490	0.0435	0.724	373
	336,400	0.2642	No. 4/0	19 × 0.1330	3,700	6,350	0.0515	0.657	316
	300,000	0.2356	188,880	19 × 0.1256	3,300	5,650	0.0578	0.621	282
	266,800	0.2094	No. 3/0	7 × 0.1953	2,940	5,040	0.0648	0.586	251
No. 4/0	211,600	0.1662	No. 2/0	7 × 0.1740	2,330	3,970	0.0816	0.522	199
No. 3/0	167,805	0.1318	No. 1/0	7 × 0.1548	1,845	3,180	0.1026	0.464	158
No. 2/0	133,079	0.1045	No. 1	7 × 0.1380	1,465	2,520	0.1294	0.414	125
No. 1/0	105,534	0.0829	No. 2	7 × 0.1228	1,160	1,990	0.1639	0.368	99.2
No. 1	83,694	0.0657	No. 3	7 × 0.1093	920	1,580	0.2070	0.328	78.6
No. 2	66,373	0.0521	No. 4	7 × 0.0975	730	1,250	0.2610	0.293	62.4
No. 3	52,634	0.0413	No. 5	7 × 0.0868	580	992	0.3291	0.258	49.5
No. 4	41,742	0.0328	No. 6	7 × 0.0772	460	786	0.4150	0.232	39.2

TABLE 90.—PHYSICAL CHARACTERISTICS—A.C.S.R.
(By courtesy of the Aluminum Company of America)

Size of conductor		Copper equivalent based upon copper 97 per cent, aluminum 61 per cent	Stranding		Outside diameter, inches	Per cent weight		Per cent area		Elastic limit, pounds	Ultimate strength, pounds	Weight in pounds per mile
Circular mils aluminum	A.w.g. or B. & S.		Aluminum	Steel		Aluminum	Steel	Aluminum	Steel			
1,590,000		1,000,000	54 × 0.1716	7 × 0.1716	1.544	72.8	27.2	88.5	11.5	38,500	55,900	10,808
1,510,500		950,000	54 × 0.1673	7 × 0.1673	1.506	72.8	27.2	88.5	11.5	36,600	53,200	10,291
1,431,000		900,000	54 × 0.1628	7 × 0.1628	1.465	72.8	27.2	88.5	11.5	34,700	50,300	9,722
1,351,500		850,000	54 × 0.1582	7 × 0.1582	1.424	72.8	27.2	88.5	11.5	32,700	47,400	9,177
1,272,000		800,000	54 × 0.1535	7 × 0.1535	1.382	72.8	27.2	88.5	11.5	30,800	44,600	8,643
1,192,500		750,000	54 × 0.1486	7 × 0.1486	1.337	72.8	27.2	88.5	11.5	28,800	41,900	8,100
1,113,000		700,000	54 × 0.1436	7 × 0.1436	1.292	72.8	27.2	88.5	11.5	26,900	39,000	7,561
1,033,500		650,000	54 × 0.1384	7 × 0.1384	1.246	72.8	27.2	88.5	11.5	25,000	36,300	7,030
954,000		600,000	54 × 0.1329	7 × 0.1329	1.196	72.8	27.2	88.5	11.5	23,100	33,500	6,490
900,000		566,000	54 × 0.1291	7 × 0.1291	1.162	72.8	27.2	88.5	11.5	21,800	31,600	6,120
874,500		550,000	54 × 0.1273	7 × 0.1273	1.146	72.8	27.2	88.5	11.5	21,200	30,700	5,940
795,000		500,000	54 × 0.1214	7 × 0.1214	1.093	72.8	27.2	88.5	11.5	19,250	27,950	5,407
715,500		450,000	54 × 0.1151	7 × 0.1151	1.036	72.8	27.2	88.5	11.5	17,360	25,200	4,857
666,600		419,000	54 × 0.1111	7 × 0.1111	1.000	72.8	27.2	88.5	11.5	16,180	23,430	4,530
636,000		400,000	54 × 0.1085	7 × 0.1085	0.977	72.8	27.2	88.5	11.5	15,400	22,300	4,320
605,000		380,500	54 × 0.1059	7 × 0.1059	0.953	72.8	27.2	88.5	11.5	14,675	21,270	4,113
556,500		350,000	30 × 0.1362	7 × 0.1362	0.953	59.9	40.1	81.1	18.9	19,370	26,800	4,595
556,500		350,000	26 × 0.1463	7 × 0.1090	0.912	70.0	30.0	87.0	13.0	14,600	20,950	3,938
518,000		326,000	42 × 0.1111	19 × 0.1111	1.000	43.5	56.5	68.9	31.1	24,000	33,000	5,891
500,000		314,500	30 × 0.1291	7 × 0.1291	0.904	59.9	40.1	81.1	18.9	17,400	24,080	4,135
477,000		300,000	30 × 0.1261	7 × 0.1261	0.883	59.9	40.1	81.1	18.9	16,600	23,000	3,944

477,000		0.3746	300,000	26 × 0.1355	7 × 0.1010	0.845	70.0	30.0	87.0	13.0	12,520	17,950	3,375
397,500		0.3122	250,000	30 × 0.1151	7 × 0.1151	0.806	59.9	40.1	81.1	18.9	13,800	19,170	3,284
397,500		0.3122	250,000	26 × 0.1236	7 × 0.0921	0.771	70.0	30.0	87.0	13.0	10,430	14,960	2,812
336,400		0.2642	0000	30 × 0.1059	7 × 0.1059	0.741	59.9	40.1	81.1	18.9	11,715	16,200	2,783
336,400		0.2642	0000	26 × 0.1138	7 × 0.0848	0.710	70.0	30.0	87.0	13.0	8,840	12,660	2,384
300,000		0.2356	188,800	30 × 0.1000	7 × 0.1000	0.700	59.9	40.1	81.1	18.9	10,440	14,430	2,481
300,000		0.2356	188,800	26 × 0.1074	7 × 0.0800	0.670	70.0	30.0	87.0	13.0	7,875	11,280	2,122
266,800		0.2095	000	26 × 0.1013	7 × 0.0755	0.632	70.0	30.0	87.0	13.0	7,000	10,040	1,885
266,800		0.2095	000	6 × 0.2108	7 × 0.0705	0.633	72.8	27.2	88.5	11.5	6,470	9,385	1,811
211,800	0000	0.1662	00	6 × 0.1880	1 × 0.1880	0.564	67.6	32.4	85.7	14.3	5,940	8,435	1,556
167,806	000	0.1318	0	6 × 0.1670	1 × 0.1670	0.501	67.6	32.4	85.7	14.3	4,690	6,660	1,227
133,077	00	0.1045	1	6 × 0.1490	1 × 0.1490	0.447	67.6	32.4	85.7	14.3	3,730	5,300	977
105,535	0	0.0829	2	6 × 0.1327	1 × 0.1327	0.398	67.6	32.4	85.7	14.3	2,960	4,200	776
83,693	1	0.0657	3	6 × 0.1182	1 × 0.1182	0.355	67.6	32.4	85.7	14.3	2,355	3,340	617
66,371	2	0.0521	4	6 × 0.1052	1 × 0.1052	0.316	67.6	32.4	85.7	14.3	1,860	2,660	488
52,635	3	0.0413	5	6 × 0.0938	1 × 0.0938	0.281	67.6	32.4	85.7	14.3	1,480	2,100	387
41,741	4	0.0328	6	6 × 0.0834	1 × 0.0834	0.250	67.6	32.4	85.7	14.3	1,170	1,665	306
33,102	5	0.0260	7	6 × 0.0743	1 × 0.0743	0.223	67.6	32.4	85.7	14.3	930	1,315	243
26,251	6	0.0206	8	6 × 0.0661	1 × 0.0661	0.198	67.6	32.4	85.7	14.3	735	1,045	192

TABLE 91.—DIRECT-CURRENT AND 60-CYCLE ALTERNATING-CURRENT RESISTANCE PER MILE—A.C.S.R.
(By courtesy of the Aluminum Company of America)

Size of conductor		Number of layers of aluminum over steel core	Copper equivalent, circular mils or A. w. g. based upon copper 97 per cent, aluminum. 61 per cent	Direct-current resistance in ohms per mile at 0 amp. and 25°C.	60-cycle a.c., resistance, ohms per mile of single conductor at 25°C.							
Circular mils aluminum	American wire gage B. & S.				0 amp. per square inch	200 amp. per square inch	400 amp. per square inch	600 amp. per square inch	800 amp. per square inch	1000 amp. per square inch	1200 amp. per square inch	1400 amp. per square inch
1,590,000		3	1,000,000	0.0587	0.0591	0.0594	0.0598	0.0607	0.0619	0.0633	0.0662	0.0698
1,510,500		3	950,000	0.0618	0.0622	0.0625	0.0629	0.0637	0.0650	0.0668	0.0691	0.0726
1,431,000		3	900,000	0.0652	0.0656	0.0659	0.0663	0.0671	0.0684	0.0703	0.0724	0.0756
1,351,500		3	850,000	0.0691	0.0695	0.0698	0.0702	0.0709	0.0721	0.0740	0.0761	0.0790
1,272,000		3	800,000	0.0734	0.0738	0.0742	0.0746	0.0752	0.0764	0.0782	0.0802	0.0829
1,192,500		3	750,000	0.0783	0.0788	0.0791	0.0795	0.0801	0.0812	0.0830	0.0849	0.0874
1,113,000		3	700,000	0.0839	0.0844	0.0848	0.0852	0.0857	0.0867	0.0885	0.0903	0.0927
1,033,500		3	650,000	0.0903	0.0909	0.0913	0.0917	0.0922	0.0931	0.0945	0.0965	0.0988
954,000		3	600,000	0.0979	0.0982	0.0985	0.0989	0.0997	0.101	0.102	0.104	0.106
900,000		3	566,000	0.104	0.104	0.105	0.105	0.106	0.107	0.108	0.110	0.112
874,500		3	550,000	0.107	0.108	0.108	0.109	0.109	0.110	0.111	0.113	0.115
795,000		3	500,000	0.117	0.119	0.119	0.119	0.119	0.120	0.121	0.123	0.125
715,500		3	450,000	0.131	0.132	0.133	0.133	0.133	0.133	0.134	0.136	0.139
666,600		3	419,000	0.140	0.141	0.142	0.142	0.142	0.143	0.144	0.146	0.148
636,000		3	400,000	0.147	0.148	0.149	0.149	0.149	0.150	0.152	0.154	0.156
605,000		3	380,500	0.154	0.155	0.156	0.156	0.157	0.158	0.160	0.162	0.163
556,500		2	350,000	0.168	0.168	0.168	0.168	0.168	0.168	0.169	0.169	0.169
556,500		2	350,000	0.168	0.168	0.168	0.168	0.168	0.168	0.169	0.169	0.169

[illegible]

The direct-current resistances above are calculated values based on a conductivity of 61 per cent at 20°C. and a constant mass-temperature coefficient of resistance at 20°C. of 0.0039. The resistances are based upon the aluminum cross section only, no account being taken of the conductivity of the steel core. Two per cent has been added to allow for increase in length due to spiral stranding. No allowance has been made for increased length due to sag, when the conductors are suspended. The alternating-current resistances are based on actual tests made on various sizes of cable at various current densities.

TABLE 92.—SELF-INDUCTION, STRANDED COPPER CONDUCTORS
[Mil-henrys per 1000 ft. of conductors (for one wire)]

(By courtesy of the National Electric Light Association, "Handbook on Overhead Line Construction," 1914)

Equilateral spacing, inches	Circular mils—size of conductor—B. & S. gage												
	500,000	450,000	400,000	350,000	300,000	250,000	0000	000	0	1	2	3	4
1	0.0758	0.0788	0.0825	0.0865	0.0912	0.0968	0.1017	0.1089	0.1159	0.1229	0.1302	0.1375	0.1444
2	0.1179	0.1209	0.1246	0.1286	0.1333	0.1389	0.1438	0.1510	0.1580	0.1650	0.1723	0.1796	0.1865
3	0.1425	0.1455	0.1492	0.1532	0.1579	0.1635	0.1684	0.1756	0.1826	0.1896	0.1969	0.2042	0.2111
4	0.1601	0.1631	0.1668	0.1708	0.1755	0.1811	0.1860	0.1932	0.2002	0.2072	0.2145	0.2218	0.2287
5	0.1737	0.1767	0.1804	0.1844	0.1891	0.1947	0.1996	0.2068	0.2138	0.2208	0.2281	0.2354	0.2424
6	0.1847	0.1877	0.1914	0.1954	0.2001	0.2057	0.2106	0.2178	0.2248	0.2318	0.2381	0.2464	0.2533
9	0.2094	0.2124	0.2161	0.2201	0.2248	0.2304	0.2353	0.2425	0.2495	0.2565	0.2638	0.2711	0.2780
12	0.2268	0.2298	0.2335	0.2375	0.2422	0.2478	0.2527	0.2599	0.2669	0.2739	0.2812	0.2885	0.2954
18	0.2515	0.2545	0.2582	0.2622	0.2669	0.2725	0.2774	0.2846	0.2916	0.2986	0.3059	0.3132	0.3201
24	0.2690	0.2720	0.2757	0.2797	0.2844	0.2900	0.2949	0.3021	0.3091	0.3161	0.3234	0.3307	0.3376
30	0.2826	0.2856	0.2893	0.2933	0.2980	0.3036	0.3085	0.3157	0.3227	0.3297	0.3370	0.3443	0.3512
36	0.2936	0.2966	0.3003	0.3043	0.3090	0.3146	0.3195	0.3267	0.3337	0.3407	0.3480	0.3553	0.3623
42	0.3031	0.3061	0.3098	0.3138	0.3185	0.3241	0.3290	0.3362	0.3432	0.3502	0.3575	0.3648	0.3717
48	0.3111	0.3141	0.3178	0.3218	0.3265	0.3321	0.3370	0.3442	0.3512	0.3582	0.3655	0.3728	0.3797
54	0.3183	0.3213	0.3250	0.3290	0.3337	0.3393	0.3442	0.3514	0.3584	0.3654	0.3727	0.3800	0.3869
60	0.3247	0.3277	0.3314	0.3354	0.3401	0.3457	0.3506	0.3578	0.3648	0.3718	0.3791	0.3864	0.3933
72	0.3358	0.3388	0.3425	0.3465	0.3512	0.3568	0.3617	0.3689	0.3759	0.3829	0.3902	0.3975	0.4044
84	0.3452	0.3482	0.3519	0.3559	0.3606	0.3662	0.3711	0.3783	0.3853	0.3923	0.3996	0.4069	0.4138
96	0.3533	0.3563	0.3600	0.3645	0.3697	0.3743	0.3792	0.3864	0.3934	0.4004	0.4077	0.4150	0.4209
108	0.3605	0.3635	0.3672	0.3712	0.3759	0.3815	0.3864	0.3936	0.4006	0.4076	0.4149	0.4222	0.4290
120	0.3669	0.3699	0.3736	0.3776	0.3823	0.3879	0.3928	0.4000	0.4070	0.4140	0.4213	0.4286	0.4362
132	0.3726	0.3756	0.3793	0.3833	0.3880	0.3936	0.3985	0.4057	0.4127	0.4197	0.4270	0.4343	0.4412
144	0.3779	0.3809	0.3846	0.3886	0.3933	0.3989	0.4038	0.4110	0.4180	0.4250	0.4323	0.4396	0.4465
156	0.3828	0.3858	0.3895	0.3935	0.3982	0.4038	0.4087	0.4159	0.4229	0.4299	0.4372	0.4445	0.4514
168	0.3873	0.3903	0.3940	0.3980	0.4027	0.4083	0.4132	0.4204	0.4274	0.4344	0.4417	0.4490	0.4559
180	0.3915	0.3945	0.3982	0.4022	0.4069	0.4125	0.4174	0.4246	0.4316	0.4386	0.4459	0.4532	0.4601

TABLE 93.—SELF-INDUCTION—A.C.S.R.
(Multiple-layer conductors—all-current densities)
(By courtesy of the Aluminum Company of America)

Circular mils or A.w.g. (B. & S.) (aluminum)	Number of wires		Inductance L in mil-henrys per mile of each conductor																				
	Alumi- num	Steel	Equilateral distance D between centers of conductors*																				
			2 ft. (0.61 M.)	2.5 ft. (0.76 M.)	3 ft. (0.91 M.)	3.5 ft. (1.07 M.)	4 ft. (1.22 M.)	5 ft. (1.52 M.)	6 ft. (1.83 M.)	7 ft. (2.13 M.)	8 ft. (2.44 M.)	9 ft. (2.74 M.)	11 ft. (3.35 M.)	13 ft. (3.96 M.)	15 ft. (4.57 M.)	17 ft. (5.18 M.)	19 ft. (5.79 M.)	21 ft. (6.40 M.)	23 ft. (7.01 M.)	25 ft. (7.62 M.)	30 ft. (9.14 M.)	35 ft. (10.67 M.)	
1,590,000	54	7	1.13	1.20	1.26	1.31	1.35	1.42	1.48	1.53	1.58	1.61	1.68	1.73	1.78	1.82	1.85	1.89	1.92	1.94	2.00	2.05	
1,510,500	54	7	1.14	1.21	1.27	1.32	1.36	1.43	1.49	1.54	1.58	1.62	1.68	1.74	1.78	1.82	1.86	1.89	1.92	1.95	2.01	2.06	
1,431,000	54	7	1.14	1.22	1.27	1.32	1.37	1.44	1.50	1.55	1.59	1.63	1.69	1.75	1.79	1.83	1.87	1.90	1.93	1.96	2.02	2.06	
1,351,500	54	7	1.15	1.22	1.28	1.33	1.37	1.45	1.51	1.55	1.60	1.64	1.70	1.75	1.80	1.84	1.88	1.91	1.94	1.96	2.02	2.07	
1,272,000	54	7	1.16	1.23	1.29	1.34	1.38	1.46	1.51	1.56	1.61	1.65	1.71	1.76	1.81	1.85	1.89	1.92	1.95	1.97	2.03	2.08	
1,192,500	54	7	1.17	1.24	1.30	1.35	1.39	1.47	1.52	1.57	1.62	1.66	1.72	1.77	1.82	1.86	1.90	1.93	1.96	1.98	2.04	2.09	
1,113,000	54	7	1.18	1.25	1.31	1.36	1.40	1.48	1.53	1.58	1.63	1.66	1.73	1.78	1.83	1.87	1.91	1.94	1.97	1.99	2.05	2.10	
1,033,500	54	7	1.19	1.26	1.32	1.37	1.41	1.49	1.55	1.60	1.64	1.68	1.74	1.79	1.84	1.88	1.92	1.95	1.98	2.00	2.06	2.11	
954,000	54	7	1.20	1.28	1.34	1.39	1.43	1.50	1.56	1.61	1.65	1.69	1.75	1.81	1.85	1.89	1.93	1.96	1.99	2.02	2.08	2.13	
900,000	54	7	1.21	1.29	1.34	1.39	1.44	1.51	1.57	1.62	1.66	1.70	1.76	1.82	1.86	1.90	1.94	1.97	2.00	2.03	2.08	2.13	
874,500	54	7	1.22	1.29	1.35	1.40	1.44	1.51	1.57	1.62	1.66	1.70	1.77	1.82	1.87	1.91	1.94	1.97	2.00	2.03	2.08	2.14	
795,000	54	7	1.23	1.30	1.36	1.41	1.46	1.53	1.59	1.64	1.68	1.72	1.78	1.84	1.88	1.92	1.96	1.99	2.02	2.05	2.10	2.15	
715,500	54	7	1.25	1.32	1.38	1.43	1.47	1.54	1.60	1.65	1.70	1.73	1.80	1.85	1.90	1.94	1.97	2.01	2.04	2.06	2.12	2.17	
666,000	54	7	1.26	1.33	1.39	1.44	1.48	1.56	1.62	1.67	1.71	1.75	1.81	1.86	1.91	1.95	1.99	2.02	2.05	2.07	2.13	2.18	
636,000	54	7	1.27	1.34	1.40	1.45	1.49	1.56	1.62	1.67	1.71	1.75	1.82	1.87	1.92	1.96	1.99	2.03	2.06	2.08	2.14	2.19	
605,000	54	7	1.28	1.35	1.41	1.46	1.50	1.57	1.63	1.68	1.72	1.76	1.83	1.88	1.93	1.97	2.00	2.03	2.06	2.09	2.15	2.20	
556,500	30	7	1.28	1.35	1.41	1.46	1.50	1.57	1.63	1.68	1.72	1.76	1.83	1.88	1.93	1.97	2.00	2.03	2.06	2.09	2.15	2.20	
556,500	26	7	1.29	1.36	1.42	1.47	1.51	1.59	1.65	1.69	1.74	1.78	1.84	1.89	1.94	1.98	2.02	2.05	2.08	2.10	2.16	2.21	
518,000	42	19	1.26	1.33	1.39	1.44	1.48	1.56	1.62	1.66	1.71	1.75	1.81	1.86	1.91	1.95	1.99	2.02	2.05	2.07	2.13	2.18	
500,000	30	7	1.29	1.37	1.42	1.47	1.52	1.59	1.65	1.70	1.74	1.78	1.84	1.90	1.94	1.98	2.02	2.05	2.08	2.11	2.17	2.21	
477,000	30	7	1.30	1.37	1.43	1.48	1.52	1.60	1.66	1.70	1.75	1.79	1.85	1.90	1.95	1.99	2.03	2.06	2.09	2.12	2.17	2.22	
477,000	26	7	1.32	1.39	1.45	1.50	1.54	1.61	1.67	1.72	1.76	1.80	1.87	1.92	1.97	2.01	2.04	2.07	2.10	2.13	2.19	2.24	
397,500	30	7	1.33	1.41	1.46	1.51	1.56	1.63	1.69	1.74	1.78	1.82	1.88	1.94	1.98	2.02	2.06	2.09	2.12	2.15	2.21	2.25	
397,500	26	7	1.35	1.42	1.48	1.53	1.57	1.64	1.70	1.75	1.79	1.83	1.90	1.95	2.00	2.04	2.07	2.11	2.13	2.16	2.22	2.27	

* See footnote on p. 589.

TABLE 93.—SELF-INDUCTION—A.C.S.R. (Continued)

Circular mills (B. & S.) (aluminum)	Number of wires		Copper equivalent, circular mills or A.w.g. based upon copper 97 per cent, aluminum 61 per cent	Inductance <i>L</i> in mil-henrys per mile of each conductor																					
	Alumi- num	Steel		Equilateral distance <i>D</i> between centers of conductors*																					
				2 ft. (0.61 M.)	2.5 ft. (0.76 M.)	3 ft. (0.91 M.)	3.5 ft. (1.07 M.)	4 ft. (1.22 M.)	5 ft. (1.52 M.)	6 ft. (1.83 M.)	7 ft. (2.13 M.)	8 ft. (2.44 M.)	9 ft. (2.74 M.)	11 ft. (3.35 M.)	13 ft. (3.96 M.)	15 ft. (4.57 M.)	17 ft. (5.18 M.)	19 ft. (5.79 M.)	21 ft. (6.40 M.)	23 ft. (7.01 M.)	25 ft. (7.62 M.)	30 ft. (9.14 M.)	35 ft. (10.67 M.)		
336,400	30	7	0000	1.36	1.43	1.49	1.54	1.58	1.66	1.72	1.76	1.81	1.85	1.91	1.96	2.01	2.05	2.09	2.12	2.15	2.17	2.23	2.28	2.35	
336,400	26	7	0000	1.38	1.45	1.51	1.56	1.60	1.67	1.73	1.78	1.82	1.86	1.93	1.98	2.03	2.07	2.10	2.13	2.16	2.19	2.25	2.30	2.38	
300,000	30	7	188,800	1.38	1.45	1.51	1.56	1.60	1.68	1.74	1.78	1.83	1.87	1.93	1.98	2.03	2.07	2.11	2.14	2.17	2.19	2.25	2.30	2.38	
300,000	26	7	188,800	1.40	1.47	1.53	1.58	1.62	1.69	1.75	1.80	1.84	1.88	1.95	2.00	2.05	2.09	2.12	2.15	2.18	2.21	2.27	2.32	2.39	
266,800	26	7	000	1.41	1.48	1.54	1.59	1.64	1.71	1.77	1.82	1.86	1.90	1.96	2.02	2.06	2.10	2.14	2.17	2.20	2.23	2.28	2.33	2.39	

Single-layer conductors—current density 0 amp. per square inch																								
266,800	6	7	000	1.46	1.53	1.59	1.64	1.68	1.76	1.81	1.86	1.91	1.94	2.01	2.06	2.11	2.15	2.19	2.22	2.25	2.27	2.33	2.38	2.45
0000	6	1	00	1.61	1.68	1.74	1.79	1.84	1.91	1.97	2.02	2.06	2.10	2.16	2.22	2.26	2.30	2.34	2.37	2.40	2.43	2.48	2.53	2.58
000	6	1	0	1.66	1.73	1.79	1.84	1.88	1.95	2.01	2.06	2.10	2.14	2.20	2.26	2.30	2.35	2.41	2.44	2.47	2.52	2.57	2.63	2.68
00	6	1	1	1.69	1.77	1.82	1.87	1.92	1.99	2.05	2.10	2.14	2.18	2.24	2.30	2.34	2.38	2.42	2.45	2.48	2.51	2.57	2.61	2.66
0	6	1	2	1.73	1.80	1.86	1.91	1.95	2.02	2.08	2.13	2.18	2.21	2.28	2.33	2.38	2.42	2.45	2.49	2.52	2.54	2.60	2.65	2.70
0	6	1	3	1.76	1.83	1.89	1.94	1.99	2.06	2.12	2.17	2.21	2.25	2.31	2.37	2.41	2.45	2.49	2.52	2.55	2.58	2.63	2.68	2.73
2	6	1	4	1.79	1.87	1.93	1.97	2.02	2.09	2.15	2.20	2.24	2.28	2.34	2.40	2.44	2.48	2.52	2.55	2.58	2.61	2.67	2.72	2.77
3	6	1	5	1.83	1.90	1.96	2.01	2.06	2.12	2.18	2.23	2.27	2.31	2.38	2.43	2.48	2.52	2.55	2.58	2.61	2.64	2.70	2.75	2.80
4	6	1	6	1.86	1.93	1.99	2.04	2.08	2.16	2.21	2.26	2.31	2.34	2.41	2.46	2.51	2.55	2.59	2.62	2.65	2.67	2.73	2.78	2.83
5	6	1	7	1.89	1.97	2.02	2.07	2.12	2.19	2.25	2.30	2.34	2.38	2.44	2.50	2.54	2.58	2.62	2.65	2.68	2.71	2.77	2.81	2.86
6	6	1	8	1.93	2.00	2.06	2.11	2.15	2.22	2.28	2.33	2.38	2.41	2.48	2.53	2.58	2.62	2.65	2.69	2.72	2.74	2.80	2.85	2.90

* See footnote on p. 589.

Single-layer conductors—current density 600 amp. per square inch

	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
266,800	1.51	1.58	1.64	1.69	1.73	1.80	1.86	1.91	1.95	1.99	2.06	2.11	2.16	2.20	2.23	2.26	2.29	2.32	2.35	2.38	2.41	2.44	2.47	2.50	2.53	2.56	2.59	2.62	2.65	2.68	2.71	2.74	2.77	2.80	2.83	2.86	2.89	2.92	2.95	2.98	3.01	3.04	3.07	3.10	3.13	3.16	3.19	3.22	3.25	3.28	3.31	3.34	3.37	3.40	3.43	3.46	3.49	3.52	3.55	3.58	3.61	3.64	3.67	3.70	3.73	3.76	3.79	3.82	3.85	3.88	3.91	3.94	3.97	4.00	4.03	4.06	4.09	4.12	4.15	4.18	4.21	4.24	4.27	4.30	4.33	4.36	4.39	4.42	4.45	4.48	4.51	4.54	4.57	4.60	4.63	4.66	4.69	4.72	4.75	4.78	4.81	4.84	4.87	4.90	4.93	4.96	4.99	5.02	5.05	5.08	5.11	5.14	5.17	5.20	5.23	5.26	5.29	5.32	5.35	5.38	5.41	5.44	5.47	5.50	5.53	5.56	5.59	5.62	5.65	5.68	5.71	5.74	5.77	5.80	5.83	5.86	5.89	5.92	5.95	5.98	6.01	6.04	6.07	6.10	6.13	6.16	6.19	6.22	6.25	6.28	6.31	6.34	6.37	6.40	6.43	6.46	6.49	6.52	6.55	6.58	6.61	6.64	6.67	6.70	6.73	6.76	6.79	6.82	6.85	6.88	6.91	6.94	6.97	7.00	7.03	7.06	7.09	7.12	7.15	7.18	7.21	7.24	7.27	7.30	7.33	7.36	7.39	7.42	7.45	7.48	7.51	7.54	7.57	7.60	7.63	7.66	7.69	7.72	7.75	7.78	7.81	7.84	7.87	7.90	7.93	7.96	7.99	8.02	8.05	8.08	8.11	8.14	8.17	8.20	8.23	8.26	8.29	8.32	8.35	8.38	8.41	8.44	8.47	8.50	8.53	8.56	8.59	8.62	8.65	8.68	8.71	8.74	8.77	8.80	8.83	8.86	8.89	8.92	8.95	8.98	9.01	9.04	9.07	9.10	9.13	9.16	9.19	9.22	9.25	9.28	9.31	9.34	9.37	9.40	9.43	9.46	9.49	9.52	9.55	9.58	9.61	9.64	9.67	9.70	9.73	9.76	9.79	9.82	9.85	9.88	9.91	9.94	9.97	10.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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Single-layer conductors—current density 1200 amp. per square inch

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By single-layer conductors is meant conductors with one layer of aluminum over the steel core. By multiple-layer conductors is meant conductors with two or more layers of aluminum over the steel core.

The inductance values of the table are based upon actual tests on various sizes of cable at various current densities at 1-ft. spacing. The inductances at other spacings were calculated from those at 1-ft. spacing by means of the fundamental inductance formula.

The inductance L' for any spacing D' not given in the table is equal to the inductance L at the next smaller spacing D given in the table plus the quantity $0.74113 \log_{10} D'/D$. Thus $L' = L + 0.74113 \log_{10} D'/D$. Or the inductance in millihenrys to be added to that at the next smaller spacing may be taken from table below.

D'/D	1.05	1.10	1.15	1.20	1.25	1.30	1.35	1.40	1.45	1.50	1.55	1.60	1.65	1.70	1.75	1.80	1.85	1.90	1.95	2.00
$L +$	0.016	0.031	0.045	0.059	0.072	0.084	0.097	0.108	0.120	0.131	0.141	0.151	0.161	0.171	0.180	0.189	0.198	0.207	0.215	0.223

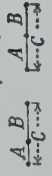
* For any three-phase arrangement of conductors $D = \sqrt[3]{ABC}$, (16.) This resolves itself into $D = A$, B or C for symmetrical triangular spacing and into $D = 1.26 A$ or B for regular flat spacing, it being immaterial whether the conductors are in a horizontal or a vertical plane.

Triangular Spacing



Unsymmetrical Symmetrical

Flat Spacing



Unsymmetrical Symmetrical

MECHANICAL FEATURES—DESIGN OF TRANSMISSION SPANS

Notation

- v = velocity in feet per second.
 V = actual wind velocity in miles per hour.
 c = velocity in miles per hour of the centers of the cups of the standard U. S. Weather Bureau anemometer.
 p = wind pressure in pounds per square foot.
 g = acceleration due to gravity = 32.16 ft. per second per second.
 B = barometric pressure in inches of mercury.
 D = external diameter of the sleet-covered wire.
 d = diameter of the wire in inches.
 a = cross-sectional area of wire in square inches.
 W = resultant weight in pounds per linear foot.
 w_w = weight of wire in pounds per linear foot.
 w_p = wind pressure on wire in pounds per linear foot.
 w_i = weight of ice in pounds per linear foot (assumed as 0.4 lb. per square inch of cross-sectional area 1 ft. long).
 N = span in feet.
 H = pull or horizontal component at lowest point, in pounds.
 S = sag in feet.
 T = tension in wire at support, in pounds.
 L = length of wire in feet.
 A = parameter = H/W .
 α = angle between tangent to curve and horizontal.
 z_0 = length of curve from lowest point to point x, y .
 e = 2.71828, base of Napierian logarithms.
 ϕ = the "Gudermannian" of $N/2A$.
 K = T/WN , or unit pull at support for unit load.
 l = L/N , or unit length of wire span.
 C = coefficient of linear expansion in degrees Fahrenheit.
 E = modulus of elasticity in pounds per square inch.

Forces Acting on the Wire Span.—The forces to be considered in transmission-line design are those due to wind pressure and to weight, not only of the structure itself but also of ice in the form of sleet adhering to the conductors.

Wind Pressure.—The pressure due to wind varies with the velocity and direction of the wind, the rapidity with which it changes velocity and direction, to some extent with the barometric pressure and temperature, and decidedly with the size, shape, and location of the surfaces exposed to the wind.

Theoretically, at a fixed temperature and barometer, the static pressure p necessary to produce a given velocity v can be computed by equating the potential energy of the air column to the kinetic energy of the motion it produces, or $p = wh = wv^2/2g$. Using the weight of air as 0.0765 lb. per cubic foot at a pressure of 760 mm. (or barometric pressure at sea level at latitude 45° and at a temperature of 15°C . or 59°F .), the relation of wind velocity and static pressure is

$$p = 0.00256V^2 \quad (17)$$

where p is the pressure in pounds per square foot and V the actual wind velocity in miles per hour.

The dynamic effect of suddenly and completely reducing the forward velocity of the wind to zero, as is the case when a flat surface of infinite size is placed normal to the wind, is also capable of a definite theoretical solution, based on the second law of Newton, that the product of force and time equals mass times velocity change. Considering the time as 1 sec. and the unit of area 1 sq. ft., under the same condition of temperature and barometer as before, $p = \frac{wp}{g} (v - v_0)$ which reduces, v_0 being zero, to

$$p = 0.00512V^2 \quad (18)$$

or twice the pressure of Eq. (16).

With the temperature at 0°C. or 32°F. the value of the coefficient due to the greater density of air would increase to 0.0027 and 0.0054. The U. S. Weather Bureau uses a multiplier of $B/30$ to include the effect of barometric changes in their formula, which is $p = 0.004 V^2 B/30$,* where B is the barometric pressure in inches of mercury. The Smeaton-Rouse formula, $p = 0.005V^2$ is based, according to Fleming, on a "slender and erroneous foundation."† Experiments of M. Eiffel at the Eiffel Tower indicated a pressure of $0.0032V^2$.‡ Other experimental formulas indicate a variation between the limits of 0.0032 and $0.004V^2$, all being based on flat plates of moderate size. Fleming concludes that $0.004V^2$ is a safe and possibly rather high value.

The latest experiments have recently been made by the U. S. Bureau of Standards,¹ the results being tabulated as coefficients to be applied to the static pressure at 15°C. previously given as $0.00256V^2$. The maximum coefficient obtained was 2 on a "rectangular flat plate of infinite length," which is in accordance with the theoretical $0.00512V^2$ for this condition and is recommended by the U. S. Bureau of Standards as applicable to radio towers and bridge girders. The coefficient for a rectangular prism of proportion 1:1:5 and applicable to tall buildings was 1.6, which is equivalent to $p = 0.0041 V^2$, agreeing with the formula of the U. S. Weather Bureau and the conclusion reached by Fleming. For a cylinder of the same proportions the coefficient was 0.8, equivalent to $p = 0.00204V^2$ applicable to chimneys and standpipes, and for a square flat plate a coefficient of 1.1 or $p = 0.00286V^2$ was found and

* "Anemometry," U. S. Weather Bureau, *Bull.* 364, 1907.

† "Wind Bracing in Industrial and Many-storied Buildings," *Jour. Boston Soc. Civil Eng.*, 1926.

‡ MERRIMAN, "American Civil Engineers' Pocketbook," p. 493.

¹ DRYDEN, HUGH L., and GEORGE C. HILL, "Wind Pressure on Structures." U. S. Bureau of Standards, *Paper* 523, Apr. 3, 1926.

considered applicable to square signboards, agreeing fairly well with other experimental values, such as those of Eiffel.

The maximum velocity of the wind that is used for ordinary purposes of design can be determined based on the maximum pressure of 30 lb. per square foot generally used for buildings. Inserting this value in the formula $p = 0.0041 V^2$, the resulting velocity is $85\frac{1}{2}$ miles per hour, which is classed by Smeaton and Rouse as a hurricane. They classify 60 miles per hour as a great storm and 100 miles per hour as a "hurricane that tears up trees, carries buildings before it," etc. Under ordinary conditions of exposure a velocity of 80 or 90 miles per hour would be ample for design purposes, but in very exposed locations and very high or important structures a velocity even greater than this may be warranted.

The records of the U. S. Weather Bureau show that only a few stations in the country, in exposed locations on the sea coast, the Great Lakes, or the plains, have recorded velocities greater than 90 miles per hour. The recorded velocity is the average over a period of 5 min., and the momentary velocity may be much higher as "the wind normally consists of a series of gusts, the speed and direction of which vary within wide limits."¹ On the other hand, the recorded velocity is not the actual velocity, but three times the velocity of the cups of the standard anemometer. The Weather Bureau *Bulletin* gives the relation $\log V = 0.509 + 0.9012 \log c$ (where c is the velocity of the centers of the cups in miles per hour) to express the relation between the cup velocity and the true wind velocity but notes that for indicated velocities over 50 or 60 miles per hour the formula is not dependable but much more accurate than the uncorrected or indicated velocity. Table 94 shows the relation between the true, cup, and indicated velocities and also gives the pressures in accordance with several of the different formulas previously noted. It will be seen that there is a marked difference between the actual and

TABLE 94.—WIND VELOCITIES AND PRESSURES

True velocity, miles per hour, V	Cup velocity, miles per hour	Recorded velocity, miles per hour	Pressures (at 15° C.)		
			Wires $p = 0.00256 V^2$ (static)	Bridge girders and radio towers U. S. Bureau of standards $p = 0.00512 V^2$ (dynamic)	Tall buildings U. S. Bureau of standards $p = 0.0041 V^2$ (dynamic)
40	16.3	49.0	4.1	8.2	6.5
50	20.9	62.8	6.4	12.8	10.2
60	25.6	76.8	9.2	18.4	14.8
70	30.4	91.2	12.5	25.1	20.1
80	35.2	105.6	16.4	32.8	26.3
90	40.2	120.5	20.8	41.5	33.2
100	45.1	135.4	25.6	51.2	41.0

¹ "Wind Pressure on Structures."

recorded velocity. Owing to the fact that the latter is an average and not a maximum value, the author believes that the actual maximum velocity on which design should be based can be taken as equal to the uncorrected values of the recorded maximum.

Sleet and Ice Loads.—Usually sleet storms, and not wind pressure alone, produce the greatest loading to which transmission structures will be subjected, and the effect of both, in combination or separately, must be considered. Sleet storms accompanied by winds of high velocity are frequent in the northeastern section of the United States and occasional in other portions of the country. Sleet deposits form usually less than $\frac{1}{4}$ in. in radial thickness on transmission wires; frequently up to $\frac{1}{2}$ in. thickness, less frequently up to 1 in. inch in thickness, and sometimes over 1 in. in thickness. In exposed locations from about 2* to 3 in.† has been observed.

The wind velocity accompanying these storms is rarely the maximum recorded velocity of the station and seldom exceeds 60 miles per hour. At Nantucket, however, a storm with sleet deposit of $1\frac{3}{4}$ in. occurred with an accompanying recorded velocity of 83 miles per hour with the temperature at 27°F.‡

Combined Wind and Ice Loading Requirements.—The joint committee on Overhead Line Construction of the National Electric Light Association has recommended loadings for the design of conductors as follows,

Class A: no ice, 15-lb. wind pressure at 0°F.

Class B: $\frac{1}{2}$ -in. ice, 8-lb. wind pressure at 0°F.

Class C: $\frac{3}{4}$ -in. ice, 11-lb. wind pressure at 0°F.

and further recommends that the wire shall be so strung that, when the specified loading occurs, the stress in the conductor shall not exceed one-half the ultimate strength of the material.

For hard-drawn copper, the material which the joint committee had in mind, the elastic limit is about 55 per cent of the ultimate strength, and in wire spans the allowed stress according to the specification is then 90 per cent of the elastic limit. If this figure of 90 per cent of the elastic limit was used as the allowable stress in the conductor at the assumed maximum loading condition, a uniform basis for all materials would be established.

Class A loading applies to regions outside the sleet-storm belt. Class B is the ordinary average loading for the sleet-storm belt and is illustrated by Fig. 260, while class C applies to the more exposed localities. The

* The transmission line of the New England Power Company over the Berkshire range, between their station 5 and North Adams, Mass.

† The pole line of the Turners Falls Power and Electric Company in Ashfield, Mass.

‡ "Handbook of Overhead Line Construction," Nat. Electric Light Assoc., p. 779, 1914.

decision as to the proper loading should depend entirely on the climatic conditions in the territory through which the line is to run. No attempt will be made to define any general districts for the different loadings, as requirements vary greatly even within a limited area. Other specifications for loading are in use, but the above are the common basis for design, modified to suit local conditions.

For wind pressure on wires, the Bureau of Standards coefficient of 0.00204 at 15°C., which corrected to 0°F. gives approximately $p = 0.0023V^2$, agrees closely with the value of $p = 0.0025V^2$ used by Coombs¹

and Fleming. Taking the usual maximum of 60 miles per hour indicated velocity during sleet storms as corresponding to class B loading conditions, the pressure calculated by the two formulas would be 5.30 and 5.76 lb. per square foot for the true velocity under steady wind conditions. An allowance of 50 per cent, which seems a reasonable amount for the variation in intensity of the wind not recorded by the anemometer, will check approximately the 8-lb. pressure, which also seems justified by fairly long use, for the conditions to which it applies.

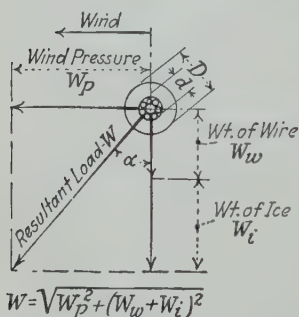


FIG. 260.—Resultant load on wire.

The same conditions that produce the wind load on the span should be used to figure that on the tower. The U. S. Weather Bureau formula gives for this condition 9.2 lb.; the Bureau of Standards for tall buildings, correcting again to 0°F., gives 11.6 lb., the formula used by Coombs ($0.0042V^2$) gives 9.7 lb. Adding 50 per cent for wind gusts gives 13.8, 17.4, and 14.6 lb. per square foot, respectively, or an average of nearly 16 lb., which should be applied to both faces of the tower as they are so separated as to be equally exposed to wind pressure. For class A and C loadings the equivalent load on the tower will be 30 and 22 lb. per square foot, or double the pressure on the wire.

High wind alone or combined with a moderate amount of sleet on the conductor may produce a greater local stress (as in the case of the web members on the transverse faces of the tower) than the recommended loading conditions. All possible cases which may produce maximum stress in any member should, therefore, be investigated for standard tower structures, which are designed for a small angle, usually 5 deg. The more important special structures should be designed for more severe loading conditions than standard line structures somewhat in the proportion that they approach radio towers in height and exposure. The general practice in the design of steel transmission structures is to increase the loading with the kilowatt capacity of the line, thus increasing

¹ COOMBS, R. D., "Poles and Tower Lines," 1916.

the reliability of service and safeguarding the capital invested in the line. The largest pressure used to date (known to the author) in standard tower-line designs are 40 and 25 lb. per square foot for tower and bare wire, respectively, for 220,000-volt transmission.

In conclusion, the use of the actual maximum recorded velocities as true velocities without correction, or the use of the true velocity corresponding to the observed maximum velocity with an addition of 50 per cent for wind gusts in the formula of the Weather Bureau, $0.004V^2$ for standard towers and the general formula $0.0025V^2$ for wires, is in accordance with both present practice and experimental data. In the design of transmission structures, wind is one of the principal forces to be resisted, not a secondary one, as with most structures, and this fact should influence the designer in his choice of loading.

Action of Wind and Ice Loading on the Conductor.—Considering a unit length of the conductor, we have these external forces to consider; two which act vertically, the weight of the wire and the weight of the ice, and one acting horizontally, *viz.*, the pressure due to wind on the effective exposed area of the ice-covered wire.

The resultant of these forces is the total load per unit length of wire, as shown in Fig. 260, and, as the wire is flexible, it swings about the points of support until it reaches a state of equilibrium in the plane of the resultant load, as shown in Fig. 261.

The wire exerts a pull on the tower which can be divided into three components. If the elevation of each support is the same, these three components are:

- I. Vertical component = one-half the total weight of wire and ice.
- II. Horizontal component in a direction normal to the line = one-half the total wind pressure on the wire and ice.
- III. Horizontal component in the direction of line = the tension at the lowest point of the wire.

For unequal elevations the horizontal component is unchanged and loads I and II are divided unequally between the two supports and are directly proportional to the distance from the support to the low point of the wire.

The horizontal component (III) in the direction of the line, as shown in Fig. 262, is the most important force we have to consider in the design of both the span and the supporting structure. This horizontal component is a constant throughout the wire and is the force necessary to hold the weighted wire in equilibrium. The determination of this tension

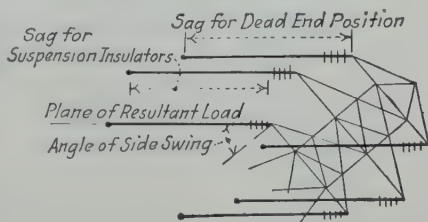


FIG. 261.—Side swing of wires in wind and plane of resultant load.

and the solution of the problem involved in the wire span can be effected approximately by assuming a parabolic curve for the wire span, which gives results with sufficient accuracy for ordinary purposes. For long spans the catenary curve, which is the theoretical curve of a non-elastic flexible chain, should be used, and, as the actual solution of the problem of changing load is as simple with the catenary as with the parabola and mathematically accurate, there seems to be no good reason for using the parabola except that it is easier to calculate the forces acting. For that reason, as well as to illustrate the points of difference between the parabola and the catenary, the derivation using a parabolic curve is given.

In Fig. 263 the wire span has been shown deflected into the plane of the resultant load, as shown by Fig. 261. Considering the portion of

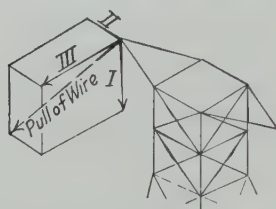


FIG. 262.—The three components of the pull of the wires.

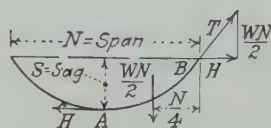


FIG. 263.—Parabolic curve for wire.

length AB , and assuming that the total weight of the wire between these points is concentrated at a distance $N/4^*$ from A , take moments about A and write the equation

$$H \times S = \frac{WN}{2} \times \frac{N}{4} = \frac{WN^2}{8} \quad (19)$$

Then
$$H = \frac{WN^2}{8S} \quad \text{or} \quad S = \frac{WN^2}{8H} \quad (20)$$

Considering Eq. (19) there are four variables, H , W , N , and S . If two of these are constant, the general relation between the two remaining variables can be determined. Thus, (1) with W and N constant, the tension varies inversely as the sag; (2) with N and S constant, the tension varies directly as the weight; (3) with S and W constant, the tension varies directly as the square of the span; (4) with H and W constant, the sag varies directly as the square of the span; (5) with N and H constant, the sag varies directly as the weight.

The wire span is the simplest of all structures for a given fixed condition of loading, as it involves only axial tension; yet with changing loads

* This is only approximately true for wires but is the basis of the parabolic curve, which assumes a uniform weight per foot of span, whereas the catenary assumes a uniform weight per foot of wire.

and temperatures the length, sag, and stress also vary, and their determination is somewhat complicated.

Equation (20), together with a formula for parabolic length of wire not given, can be used to calculate but one of these conditions of length and sag for a given stress, and that one being determined, it is necessary to use the relation of coefficient of expansion and modulus of elasticity of the wire to complete the solution of the problem for any other loading or temperature conditions. As stated previously, the catenary permits as simple a solution as the parabola and is, therefore, used.

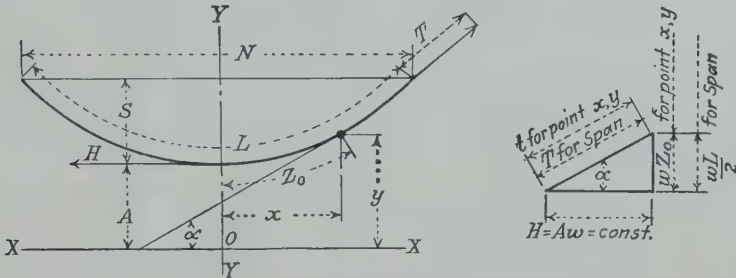


FIG. 264.—Derivation of the catenary.

Derivation

$$\begin{aligned} \frac{wz_0}{H} &= \tan \alpha, A = \frac{H}{w}, z_0 = A \tan \alpha. \quad dz_0 = A \sec^2 \alpha d\alpha. \quad \tan \alpha = \frac{dy}{dz_0}, \sin \alpha = \frac{dy}{dz_0}, \cos \alpha = \frac{dx}{dz_0} \\ \frac{dy}{dz_0} \cdot \frac{dz_0}{d\alpha} &= A \sec^2 \alpha \sin \alpha; \quad y - A = A \int_0^\alpha \frac{\sin \alpha d\alpha}{1 - \sin^2 \alpha}, \text{ then let } m = \sin \alpha, dm = \cos \alpha d\alpha. \quad dm = \\ (1 - m^2)^{\frac{1}{2}} d\alpha; \quad y - A &= A \int_0^m \frac{mdm}{\sqrt{(1 - m^2)^3}}; \quad y - A = A \left[\frac{1}{\sqrt{1 - m^2}} - 1 \right]; \quad y = \frac{A}{\cos \alpha}, \quad \frac{y}{A} = \sec \alpha \\ \frac{dx}{dz_0} \cdot \frac{dz_0}{d\alpha} &= A \sec^2 \alpha \cos \alpha. \quad x = A \int_0^\alpha \sec \alpha d\alpha, \quad x = A \log_e \tan \left(\frac{\pi}{4} + \frac{\alpha}{2} \right) \text{ then } e^{\frac{x}{A}} = \tan \left(\frac{\pi}{4} + \frac{\alpha}{2} \right) \\ e^{\frac{x}{A}} &= \frac{1 + \tan \frac{\alpha}{2}}{1 - \tan \frac{\alpha}{2}}; \quad e^{-\frac{x}{A}} = \frac{1 - \tan \frac{\alpha}{2}}{1 + \tan \frac{\alpha}{2}}; \quad \frac{1}{2} \left(e^{\frac{x}{A}} + e^{-\frac{x}{A}} \right) = \frac{1 + \tan^2 \frac{\alpha}{2}}{1 - \tan^2 \frac{\alpha}{2}} = \sec \alpha = \frac{y}{A} \\ \therefore \text{ equation of catenary curve } y &= \frac{A}{2} \left(e^{\frac{x}{A}} + e^{-\frac{x}{A}} \right) \text{ or } y = A \cosh \frac{x}{A} \text{ expressed in hyperbolic} \end{aligned}$$

functions

$$\begin{aligned} \text{The length } z_0 \text{ is found as follows } dy &= \sinh \frac{x}{A} dx, \quad z_0 = \int_0^x \sqrt{1 + \frac{dy^2}{dx^2}} dx = \int_0^x \sqrt{1 + \sinh^2 \frac{x}{A}} dx \\ &= \int_0^x \cosh \frac{x}{A} dx, \dagger \end{aligned}$$

$\therefore$ the equation of length of the catenary curve, $z_0 = A \sinh \frac{x}{A}$ §

$$* \cosh x = \frac{1}{2}(e^x + e^{-x})$$

$$\dagger d \frac{\cosh x}{dx} = \sinh x$$

$$\ddagger \cosh^2 x - \sinh^2 x = 1$$

$$\S \int \cosh x dx = \sinh x$$

Stresses in Wire Span. *The Catenary—Supports at Equal Elevations.* The catenary, as previously stated, is the curve taken by an inelas-

tic, flexible chain of uniform cross section and homogeneous material. Although the wire is elastic, the effect of changing stress on the uniformity of weight is negligible, and, therefore, for any given set of conditions the wire will hang in a true catenary. Figure 264 gives its derivation and Table 95 gives a summary of the equations developed in exponential, hyperbolic, and circular or trigonometric functions for any point on the span and for full-span values. The simplest formulas, those expressed by the circular or trigonometric functions, may be used by means of a table of Gudermannian's¹ giving the value of the angle ϕ in circular measure, which for any assumed trigonometric function gives the equivalent exponential or hyperbolic functions based upon the argument x/A or $N/2A$, respectively.

The late Madison S. Dow calculated for the Turners Falls Power and Electric Company Table 96 of unit values, after the method of Percy H. Thomas,² covering the entire range of transmission problems, which is of great assistance. For the argument $N/2A$, N being considered unity throughout, the corresponding values of the Gudermannian ϕ are given: the value A or the horizontal component of the pull in the wire for a weight of unity; the length l along the wire between supports; the pull K in the wire at the supports for a load and span of unity; and the sag λ in feet.

TABLE 95.—SUMMARY OF THE CATENARY EQUATIONS

Exponential functions	Hyperbolic functions	Circular or trigonometric functions
For point (x, y)	For point (x, y)	
$y = \frac{A}{2} \left(e^{\frac{x}{A}} + e^{-\frac{x}{A}} \right)$	$y = A \cosh \frac{x}{A}$	$y = A \sec \phi^1$
$z_0 = \frac{A}{2} \left(e^{\frac{x}{A}} - e^{-\frac{x}{A}} \right)$	$z_0 = A \sinh \frac{x}{A}$	$z_0 = A \tan \phi$
For the full span	For the full span	
Sag $S = \frac{A}{2} \left(e^{\frac{N}{2A}} + e^{-\frac{N}{2A}} \right) - A$	$S = A \left(\cosh \frac{N}{2A} - 1 \right)$	$S = A (\sec \phi - 1) = A \operatorname{exsec} \phi$
Length $L = A \left(e^{\frac{N}{2A}} - e^{-\frac{N}{2A}} \right)$	$L = 2A \sinh \frac{N}{2A}$	$L = 2A \tan \phi$

¹ Where $\phi = \operatorname{gd} \frac{x}{A}$ or $\frac{N}{2A}$.

² PIERCE, "A Short Table of Integrals."

² "Sag Calculations for Suspended Wires," *Jour. A.I.E.E.*, vol. 30, Part III, pp. 2229-2239, 1911.

TABLE 96.—UNIT VALUES OF LENGTH, PULL, AND SAG FOR THE ARGUMENT $N/2A$ AND THE GUDERMANNIAN ϕ , WHERE $N = 1$

ϕ	$\frac{N}{2A}$	A	l	K	λ
0°-10'	0.0029089	171.8863	1.00000141	171.88703	0.00072722
20	0.0058178	85.94314	1.00000564	85.94459	0.00145446
30	0.0087268	57.29478	1.00001269	57.29696	0.00218172
40	0.0116350	42.97045	1.00002256	42.97336	0.0029090
50	0.0145450	34.37608	1.00003525	34.37972	0.0036363
1°-0'	0.0174543	28.64624	1.00005076	28.65060	0.0043636
10	0.0203637	24.55349	1.00006910	24.55858	0.0050910
20	0.0232732	21.48394	1.00009027	21.48976	0.0058184
30	0.0261830	19.09636	1.00011427	19.10291	0.0065460
40	0.0290930	17.18627	1.00014110	17.19354	0.0072736
50	0.0320032	15.62344	1.00017076	15.63144	0.0080014
2°-0'	0.0349137	14.32103	1.00020325	14.32976	0.0087292
10	0.0378245	13.21895	1.00023857	13.22841	0.0094572
20	0.0407357	12.27425	1.00027672	12.28444	0.0101853
30	0.0436472	11.45549	1.00031770	11.46640	0.0109135
40	0.0465590	10.73906	1.00036151	10.75070	0.0116418
50	0.0494712	10.10689	1.00040815	10.11926	0.0123703
3°-0'	0.0523839	9.54492	1.00045763	9.55802	0.0130989
10	0.0552970	9.04208	1.00050995	9.05601	0.0138277
20	0.0582105	8.58952	1.00056511	8.60408	0.0145567
30	0.0611245	8.18003	1.00062311	8.19532	0.0152859
40	0.0640391	7.80773	1.00068395	7.82375	0.0160153
50	0.0669543	7.46778	1.00074763	7.48452	0.0167448
4°-0'	0.0698700	7.15615	1.00081416	7.17362	0.0174746
10	0.0727863	6.86943	1.00088354	6.88763	0.0182046
20	0.0757032	6.60475	1.00095577	6.62368	0.0189348
30	0.0786207	6.35965	1.00103085	6.37932	0.0196653
40	0.0815389	6.13204	1.00110879	6.15244	0.0203960
50	0.0844578	5.92012	1.00118959	5.94152	0.0211270
5°-0'	0.0873774	5.72231	1.00127325	5.74417	0.0218583
10	0.0902978	5.53724	1.00135978	5.55983	0.0225899
20	0.0932189	5.36372	1.00144918	5.38704	0.0233217
30	0.0961409	5.20070	1.00154146	5.22475	0.0240538
40	0.0990636	5.04726	1.00163662	5.07205	0.0247862
50	0.1019872	4.90258	1.00173467	4.92810	0.0255190
6°-0'	0.1049117	4.76590	1.00183561	4.79215	0.0262520
10	0.1078363	4.64142	1.0020459	4.64184	0.0277192
20	0.1107618	4.52874	1.0022681	4.53166	0.0291878
30	0.1136873	4.42636	1.0025035	4.41302	0.0306579
40	0.1166137	4.33385	1.0027475	4.32808	0.0321295
50	0.1195401	4.25091	1.00300048	4.25791	0.0336029
7°-0'	0.1224665	4.17742	1.0032737	4.20441	0.0350779
10	0.1253929	4.11392	1.0035545	4.16216	0.0365548
20	0.1283193	4.05042	1.0038471	4.13092	0.0380335
30	0.1312457	4.00000	1.0041416	4.10000	0.0395143
40	0.1341721	3.95958	1.0044380	4.06958	0.0409971
50	0.1370985	3.92916	1.0047364	4.03916	0.0424820
8°-0'	0.1399249	3.89874	1.0050348	4.00874	0.0439669
10	0.1428513	3.86832	1.0053332	3.97832	0.0454518
20	0.1457777	3.83790	1.0056316	3.94790	0.0469367
30	0.1487041	3.80748	1.0059300	3.91748	0.0484216
40	0.1516305	3.77706	1.0062284	3.88706	0.0499065
50	0.1545569	3.74664	1.0065268	3.85664	0.0513914
9°-0'	0.1574833	3.71622	1.0068252	3.82622	0.0528763
10	0.1604097	3.68580	1.0071236	3.79580	0.0543612
20	0.1633361	3.65538	1.0074220	3.76538	0.0558461
30	0.1662625	3.62496	1.0077204	3.73496	0.0573310
40	0.1691889	3.59454	1.0080188	3.70454	0.0588159
50	0.1721153	3.56412	1.0083172	3.67412	0.0603008
10°-0'	0.1750417	3.53370	1.0086156	3.64370	0.0617857
10	0.1779681	3.50328	1.0089140	3.61328	0.0632706
20	0.1808945	3.47286	1.0092124	3.58286	0.0647555
30	0.1838209	3.44244	1.0095108	3.55244	0.0662404
40	0.1867473	3.41202	1.0098092	3.52202	0.0677253
50	0.1896737	3.38160	1.0101076	3.49160	0.0692102
11°-0'	0.1926001	3.35118	1.0104060	3.46118	0.0706951
10	0.1955265	3.32076	1.0107044	3.43076	0.0721800
20	0.1984529	3.29034	1.0110028	3.40034	0.0736649
30	0.2013793	3.25992	1.0113012	3.36992	0.0751498
40	0.2043057	3.22950	1.0116000	3.33950	0.0766347
50	0.2072321	3.19908	1.0118984	3.30908	0.0781196
12°-0'	0.2101585	3.16866	1.0121968	3.27866	0.0796045
10	0.2130849	3.13824	1.0124952	3.24824	0.0810894
20	0.2160113	3.10782	1.0127936	3.21782	0.0825743
30	0.2189377	3.07740	1.0130920	3.18740	0.0840592
40	0.2218641	3.04698	1.0133904	3.15698	0.0855441
50	0.2247905	3.01656	1.0136888	3.12656	0.0870290
13°-0'	0.2277169	2.98614	1.0139872	3.09614	0.0885139
10	0.2306433	2.95572	1.0142856	3.06572	0.0899988
20	0.2335697	2.92530	1.0145840	3.03530	0.0914837
30	0.2364961	2.89488	1.0148824	3.00488	0.0929686
40	0.2394225	2.86446	1.0151808	2.97446	0.0944535
50	0.2423489	2.83404	1.0154792	2.94404	0.0959384
14°-0'	0.2452753	2.80362	1.0157776	2.91362	0.0974233
10	0.2482017	2.77320	1.0160760	2.88320	0.0989082
20	0.2511281	2.74278	1.0163744	2.85278	0.1003931
30	0.2540545	2.71236	1.0166728	2.82236	0.1018780
40	0.2569809	2.68194	1.0169712	2.79194	0.1033629
50	0.2599073	2.65152	1.0172696	2.76152	0.1048478
15°-0'	0.2628337	2.62110	1.0175680	2.73110	0.1063327
10	0.2657601	2.59068	1.0178664	2.70068	0.1078176
20	0.2686865	2.56026	1.0181648	2.67026	0.1093025
30	0.2716129	2.52984	1.0184632	2.63984	0.1107874
40	0.2745393	2.49942	1.0187616	2.60942	0.1122723
50	0.2774657	2.46900	1.0190600	2.57900	0.1137572
16°-0'	0.2803921	2.43858	1.0193584	2.54858	0.1152421
10	0.2833185	2.40816	1.0196568	2.51816	0.1167270
20	0.2862449	2.37774	1.0199552	2.48774	0.1182119
30	0.2891713	2.34732	1.0202536	2.45732	0.1196968
40	0.2920977	2.31690	1.0205520	2.42690	0.1211817
50	0.2950241	2.28648	1.0208504	2.39648	0.1226666
17°-0'	0.2979505	2.25606	1.0211488	2.36606	0.1241515
10	0.3008769	2.22564	1.0214472	2.33564	0.1256364
20	0.3038033	2.19522	1.0217456	2.30522	0.1271213
30	0.3067297	2.16480	1.0220440	2.27480	0.1286062
40	0.3096561	2.13438	1.0223424	2.24438	0.1300911
50	0.3125825	2.10396	1.0226408	2.21396	0.1315760
18°-0'	0.3155089	2.07354	1.0229392	2.18354	0.1330609
10	0.3184353	2.04312	1.0232376	2.15312	0.1345458
20	0.3213617	2.01270	1.0235360	2.12270	0.1360307
30	0.3242881	1.98228	1.0238344	2.09228	0.1375156
40	0.3272145	1.95186	1.0241328	2.06186	0.1390005
50	0.3301409	1.92144	1.0244312	2.03144	0.1404854
19°-0'	0.3330673	1.89102	1.0247296	2.00102	0.1419703
10	0.3359937	1.86060	1.0250280	1.97060	0.1434552
20	0.3389201	1.83018	1.0253264	1.94018	0.1449401
30	0.3418465	1.80000	1.0256248	1.90976	0.1464250
40	0.3447729	1.76958	1.0259232	1.87934	0.1479099
50	0.3476993	1.73916	1.0262216	1.84892	0.1493948
20°-0'	0.3506257	1.70874	1.0265200	1.81850	0.1508797
10	0.3535521	1.67832	1.0268184	1.78808	0.1523646
20	0.3564785	1.64790	1.0271168	1.75766	0.1538495
30	0.3594049	1.61748	1.0274152	1.72724	0.1553344
40	0.3623313	1.58706	1.0277136	1.69682	0.1568193
50	0.3652577	1.55664	1.0280120	1.66640	0.1583042
21°-0'	0.3681841	1.52622	1.0283104	1.63598	0.1597891
10	0.3711105	1.49580	1.0286088	1.60556	0.1612740
20	0.3740369	1.46538	1.0289072	1.57514	0.1627589
30	0.3769633	1.43496	1.0292056	1.54472	0.1642438
40	0.3798897	1.40454	1.0295040	1.51430	0.1657287
50	0.3828161	1.37412	1.0298024	1.48388	0.1672136
22°-0'	0.3857425	1.34370	1.0301008	1.45346	0.1686985
10	0.3886689	1.31328	1.0303992	1.42304	0.1701834
20	0.3915953	1.28286	1.0306976	1.39262	0.1716683
30	0.3945217	1.25244	1.0309960	1.36220	0.1731532
40	0.3974481	1.22202	1.0312944	1.33178	0.1746381
50	0.4003745	1.19160	1.0315928	1.30136	0.1761230

The actual values can then be computed for a given span and resultant weight per foot; the length of the wire, $L = lN$; the actual pull in the wire, $T = KWN$; the actual sag, $S = \lambda N$; and the horizontal component of the pull, $H = AW$.

ture, are given and the sag of the wire is to be determined. This fixes all future positions of the wire for all future conditions of loading and temperature. The proper stringing position for the wire under no-load conditions for the temperature range during stringing must then be determined based on this condition. Table 97 gives a complete example of this kind for a steel wire with the graphical part of the solution performed on Fig. 265.

The value of the elastic modulus for the wire used in the problem is somewhat high for stranded wire, as will be noted by reference to Table 86, but will serve for purposes of illustration. The solution of the design loading condition involves only the reduction of the known pull in the wire to a unit basis by dividing it by the span and resultant weight per foot of wire and determining for the value of K thus found the corresponding unit length and sag, which multiplied by the span completes the solution for the assumed loading conditions.

There are other possible conditions to be examined; for example, where the elastic modulus is small, as in the case of aluminum, the decrease in sag during cold weather with no live load may produce a greater stress in the wire than for the loading conditions assumed. It must be borne in mind that the sag figured is on a plane inclined to the vertical, although it is usual to consider it vertical due to swing. Lower sags may result with different ice-loading conditions, especially under no-load conditions at high temperatures.

The calculation of the effect of changing load conditions, which involves three unknowns, length, sag, and pull, is accomplished by the graphical solution of the simultaneous values of two equations: one, the catenary relation between the pull and length of wire for a given loading and temperature; and the other, the stress-strain relation of the wire for the same loading and temperature. As the pull of the wire is the y ordinate and the length is the x ordinate, the stress-strain curve in the proper units can also be plotted on the same coordinate system, and the point where the two curves intersect will supply the simultaneous value of length and pull in the wire to satisfy both the catenary and stress-strain relations. The only remaining unknown is the origin of the stretch-pull¹ curve, which is obtained by computing the length of the catenary at 0 stress by the formula² $T_1 l_1 / aE$ which gives the unit shortening for a wire of length l_1 with the stress reduced from T_1 to 0.

¹ To differentiate from the stress-strain curve, as in this case, total pull and total stretch in a wire of 1-ft. span are used, not unit stress as in the stress-strain curve.

² This actually involves a slight error, as T is the pull at the supports and not the average pull in the wire, which can be corrected approximately by multiplying T by a coefficient which will reduce it to the average stress. On the assumption that the average stress will be the direct average of A and K , this coefficient will be $\frac{A + K}{2K}$.

As l_1 , the unit length of the wire, at 1-ft. span is always nearly equal to unity, it is generally sufficiently accurate to consider it so, as is done in the problem. This shortening is then deducted from the unit length of wire l_1 , the remainder being the length of the wire at $T = 0$, which gives the origin of the stretch-pull curve on the x axis, as shown in Fig. 265, and a line connecting it with the point $T_1 l_1$ on the catenary will be the stretch-pull line for the particular loading considered at the given temperature.

For a different temperature condition, the increase or decrease in length is calculated by the product of the coefficient of expansion of the material and the difference in temperature between the two conditions considered or $C(T_2 - T_1)l_0$. Here again l_0 may be neglected in cases where it is nearly unity, as was done in Table 97. This new position of l_0 , adding the increase in length to that at the lower temperature, is then plotted on the $K = 0$ line, and by a stretch line parallel to the original stretch line the intersection with the catenary gives the new length and stress for the new temperature and original loading.

For a different loading W_2 the value K_1 represents a pull in the wire different from before, and the stretch line for the new loading will have a y coordinate or a K value equivalent to the stretch at K_1 and load W_1 of $K_2 = K_1 W_1 / W_2$, or inversely proportional to the ratio of resultant loads per foot for the two cases. The stretch line can then be drawn by adding to the value of l_1 the increase (or decrease) in length for the temperature change considered, plotting it against the new value of K_2 and drawing the line from that point to the value of l_0 on the $K = 0$ line, corresponding to the same temperature. Or, more simply, assume any convenient value of K , solve for T with the new loading, compute the stretch, add it to l_0 , and plot the point which connected to l_0 will give the stretch line, and the intersection with the length curve of the catenary will give the new length and stress of the wire, and the new sag can be found from the new stress. Having completely determined any two conditions of loading, a check is readily obtained by computing

Referring to Table 96, this coefficient will be as follows for values of ϕ from 0 to 20 deg., it being evident that the precision of the data does not warrant the application of this coefficient in any ordinary case.

ϕ , Degrees	Percentage Coefficient
	$\left(\frac{A + K}{2K}\right)$
2	99.97
4	99.88
6	99.72
10	99.24
15	98.30
20	96.98

the stretch due to the difference in pull and adding or subtracting the change in length due to temperature, which should check the difference in length of the two catenaries.

The diagram of Fig. 265 has been plotted in such a way as to show the nature of the stretch lines. In plotting diagrams for general use, it is frequently wasteful of space to show the $K = 0$ line, and for that reason it will be often impossible to plot the origin of the stretch-pull curves on the sheet. As the curve is assumed straight within the elastic limit, it is a simple problem of proportion to find points on the stretch-pull curve so chosen that they will fall where desired on the sheet. For example, had the limits of the diagram been $K = 1.5$ to $K = 3.0$, the value of the unit length l on the $K = 1.5$ line, for the bare-wire condition at 50°F . would be $l_{ot50} + \frac{0.00046 * \times 1.5}{3.0} = 1.00464\dagger + 0.00023 = 1.00487$.

Additional Features of Design.—Insulators in the strain position form a part of the curve, and it is customary to neglect their effect on its shape. If desired, the exact curve can be found by a combination of graphical statics and a trial-and-error method, by solving for the span and sag of the catenary between the wire clamps, assuming several values of pull and span at this point, each consistent with the proper value and direction of the pull at the supports as modified by the weight of the insulator string.

For problems similar to the example of Table 97, where the wire is to be sagged at a lighter loading and higher temperature than class B loading, when the physical characteristics of the wire are at all uncertain, the higher value of the modulus of elasticity, cross-sectional area, coefficient of linear expansion, and unit weight should be used, in order not to exceed the assumed stress at the design loading conditions.

The general method of stretch lines is applicable beyond the elastic limit, if the stretch line is plotted as the actual stress-strain curve, but if it is necessary to calculate the effect of loading the wire beyond the elastic limit, it can best be done by trial and error, assuming several values of stretch over that of the original loading, finding the corresponding pull from the stress-strain curve, and then computing the corresponding value of pull in the catenary by assuming equal changes in length from the original loading condition. The solution will be found at the intersection of the curve of catenary pull and the stress due to stretch, each plotted against the same length of wire.

The effect of increasing the load steadily beyond the elastic limit will be to increase greatly the length of the wire, and the sag will increase to such an extent that the wire will usually be on or near the ground when it is at the point of breaking, and the loading necessary to break it will be

* 0.00046 is the value of unit stretch when $K = 3.00$.

† 1.00464 is the value of unit unstressed length at 50°F .

greatly in excess of the assumed loading condition. Usually, however, except in case of sleet formation with no wind, the loading on the wire is subject to violent changes, and failure will occur at a weak point, usually at the insulator connection, before the wire is greatly extended by stress beyond the elastic limit. It is not good practice, however, to sag a wire so that it will be stressed beyond the elastic limit at the assumed loading conditions, but, where such heavy sleet accumulations occur as in the cases mentioned, it is practically impossible from an economic standpoint to design against such conditions.

In the case of the New England Power Company previously cited, the formation of sleet was so frequent and accompanied by such high winds that a much heavier cable and much heavier steel towers with half the span of the original line were installed. Further to safeguard the line at this exposed point, a sleet-thawing apparatus was installed still later

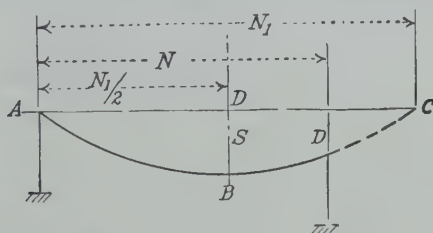


FIG. 266.—Catenary with supports at different elevations.

and, when sleet starts to form on the conductors, observers notify the station operator and the voltage is lowered from 66,000 to 6100 volts, to produce a current sufficient to clear the sleet from the conductor.

On long spans, where the tension in the wire is great, a vibration sometimes occurs like that in a bridge under steady repetition of load, which results in the eventual breaking of the wire by crystallization at the support. Changing the tension in the wire slightly may prevent this vibration. The Southern California Edison Company¹ has attached small weights or dampers to the conductor just beyond the clamp, which have proved to be satisfactory in actual use. The details of the damper and measurement of the vibration found are given in the reference noted.

Catenary with Supports at Different Elevations.—This condition is illustrated by Fig. 266, and the maximum sag and its location may be found by the use of Fig. 267. The use of this diagram will be clear from the following example:

Assuming

$$T_1 = 4000 \text{ lb.}, N = 900 \text{ ft.}, D = 50 \text{ ft.}, W = 1.5 \text{ lb.}$$

then

¹ "Eliminating Vibration in Ground Cables," *Elec. World*, vol. 87, No. 25, p. 1351, 1926.

$$K_1 = \frac{T_1}{WN} = \frac{4000}{1.5 \times 900} = 2.96$$

$$\text{and} \quad d = \frac{D}{N} = \frac{50}{900} = 0.0555$$

As shown in Fig. 267, for these values of K_1 and d ,

$$\lambda_1 = 0.0576 \text{ and } \gamma = 0.759$$

$$\text{Hence,} \quad N_1 = \frac{900}{0.759} = 1185 \text{ (AC in Fig. 266); } \frac{N_1}{2} = 592.5$$

$$\text{and} \quad S = 1185 \times 0.0576 = 68.2 \text{ ft.}$$

For changing load and temperature, it is necessary to use the method of trial and error which has been previously discussed. This involves a new position of the origin or lowest point in the wire. The position of this point (B in Fig. 266) and sag may be assumed, and the stress and length computed and compared with the change in stress for change in length for several adjacent positions, from which, by interpolation, the correct results for the exact conditions in question may be obtained.

MECHANICAL FEATURES OF SUPPORTING STRUCTURES

Types of Supporting Structures.—The choice of type of supporting structure for overhead transmission lines depends principally on the load imposed on the structure by the wire span. The three following general types are classified in accordance with their ability to carry load: (1) the tower, (2) the A- or H-frame, and (3) the pole.

1. *The tower* is generally a four-legged (very infrequently, a three-legged) structure, each leg having a separate anchorage or foundation. It differs from all the other types in that it can, without excessive cost, be made self-supporting, without guying, under any imposed load acting in any direction, as the spread of the base permits an anchorage with a weight and moment arm sufficient to resist the cantilever action of the structure under the action of wind pressure and pull in the conductor. Wooden towers have been built occasionally for special purposes on wood-pole lines where long spans were necessary, or in very soft ground. Steel towers are used almost entirely with suspension insulators on nearly all important large-capacity high-voltage lines where the loading conditions are at all severe. The standard line tower for spans of 300 to 1700 ft. is usually a bolted and galvanized structure. Riveted and painted structures are used only for special construction.

2. *A- or H-frames.*—The wood A-frame, the wood H-frame shown in Fig. 268, and the flexible steel frame as shown in Fig. 269 are two-legged structures, self-supporting under load, in a direction transverse to the line only and depend on guying or dead-end towers placed at frequent intervals (usually 5 to 10 spans apart) for stability in the direction of the

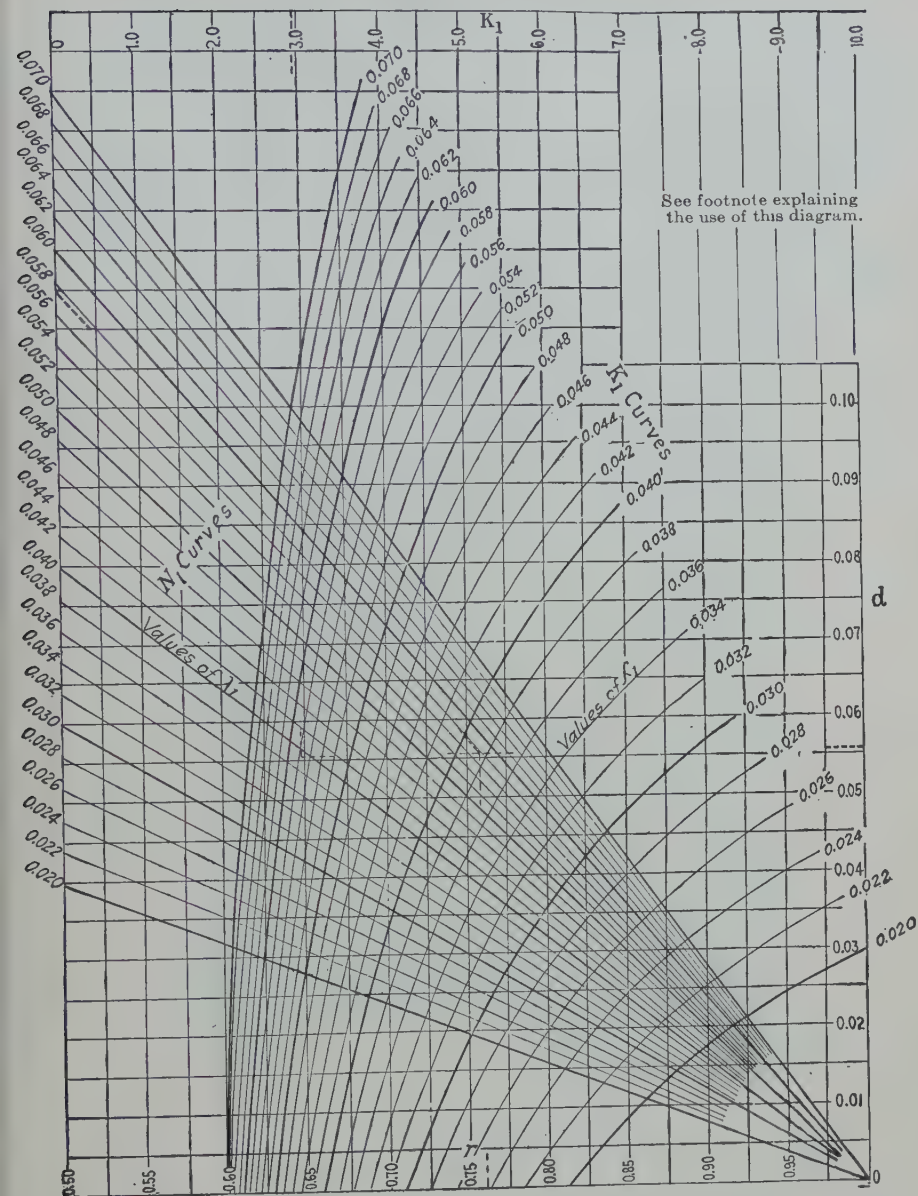


FIG. 267.—Sag-tension curves for suspended wires. Supports at different elevations.

USE OF DIAGRAM

Given: Span N ; difference in elevation D ; tension at upper support T_1 .

To Find: 1. Distance of low point of catenary from upper support. 2. Sag of wire.

Procedure: 1. Determine $K_1 = T_1/WN$, where W is unit load at specified conditions for tension T_1 .
2. Determine $d = D/N$. 3. Enter curve with these values of K_1 and d and determine λ_1 on K_1 curves. 4. Project λ_1 horizontally to corresponding λ on N_1 curves and read on the abscissas the value of γ . Compute $N_1 = N/\gamma$.

Results: Distance of low point of catenary from upper support is $N_1/2$; sag = $\lambda_1 N_1$.

Note: γ is ratio of given span to completed span of same catenary with both supports of same level = N/N_1 . $\lambda_1 = S/N_1$.

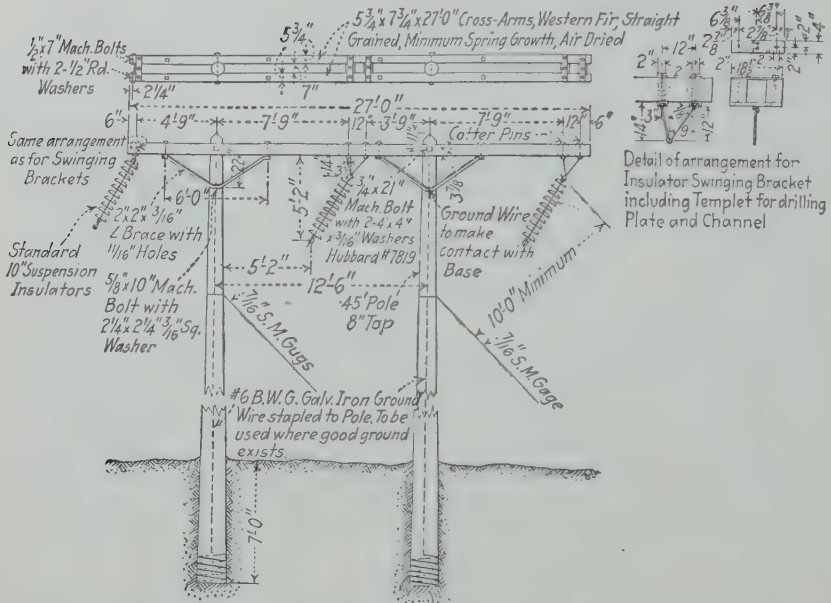


FIG. 268.—Pineville-Dix River 132-kv. wood transmission line. Pole framing for two poles, angle construction—type B. Kentucky Hydroelectric Company.

BILL OF MATERIAL

Item	Quantity	Description
1	2	45' Wood Poles—8" top
2	2	Cross-Arms $5\frac{3}{4}" \times 7\frac{3}{4}" \times 27'-0"$
3	30	Suspension Insulators
4	2	$3\frac{1}{2}" \times 21"$ Mach. Bolts with 4" thread and nut
5	4	Cross-Arm Braces—Hubbard Cat. #7956
6	20	$5\frac{1}{2}" \times 10"$ Mach. Bolts with nut
7	4	$4" \times 4" \times \frac{3}{16}"$ Sq. Washers
8	8	$2\frac{1}{4}" \times 2\frac{1}{4}" \times \frac{3}{16}"$ Sq. Washers
9	2	Swinging Insulator Brackets (Am. Bridge Co.)
10	5	$3\frac{1}{2}" \times 11"$ U Bolts with nuts, and Cotter Pins
11	5	$4"$ Channels, $18\frac{1}{2}"$ lg. drilled as shown
12	5	$4" \times 18\frac{1}{2}" \times \frac{1}{4}"$ Plates
13	5	$5\frac{3}{4}" \times 7\frac{3}{4}" \times \frac{7}{8}"$ Wood Spacers (made for $\times$ arms)
14	3	Suspension Hooks
15	3	Ball Socket Clevis
16	3	Suspension Clamps
17	150'	#6 B.W.G. galv. ground wire
18	100	Wire Staples galv. (for ground wires)
19	130'	$\frac{3}{16}"$ S. M. Guy Wire
20	4	3 Bolt Clamp for $\frac{3}{16}"$ wire
21	4	Guy Hooks—Hubbard Cat. #7584
22	4	Guy Plates—Hubbard Cat. #7575
23	2	Guy Rods $1" \times 8'-0"$ lg.
24	2	Guy Thimbles—Hubbard Cat. #7595
25	4	Lag Screws for Guy Hooks $\frac{1}{2}" \times 4"$ lg.
26	2	$4" \times 4" \times \frac{1}{2}"$ Washers for Guy Anchor

NOTE: Cross arms to be drilled in mill. All hardware to be galvanized. Complete structure to be assembled on ground including insulators, before raising.

Five (5) insulators per string to be used for 66,000 volts.

Ten (10) insulators per string to be used for 132,000 volts.

line. When the conductors break in any span, the failure of all frames of this type (if pin insulators are used and the wire does not pull loose at the tie wire) between dead-end towers or guyed frames must follow. With one conductor broken, the flexibility of the tower in the direction of the line allows the adjacent spans to be shortened and the tension to be relieved, while the stress in the remaining conductors in the broken-wire span increases to equalize the total line tension.



FIG. 269.—Steel A-frame tower and wood pole line—Taylors Falls plant, Minneapolis General Electric Company. Two 43-mile circuits, 10,000 kw., 50,000 volts.

The flexible steel tower, used for spans of 200 to 500 ft., is adapted to pin-type insulators only, as it requires support in the line direction, and it is difficult to string the wires for this reason, particularly the ground wire, which acts as a temporary support while stringing the conductors. As this type of steel frame has strength only in one direction and has practically no strength in torsion and does not adapt itself to the non-rigid foundation usual in transmission, a number of failures have resulted from its use. It may be concluded that this type of tower, while low in first cost, is not so serviceable as other similar structures like the steel-pole

or the wood H-frame, which have more strength in the direction of the line and considerably more strength in torsion.

The wood H-frame is being used successfully with wood and steel cross arms for spans of 200 to 700 ft. and with suspension insulators for high voltages and large capacities, as in Fig. 268, which shows in detail

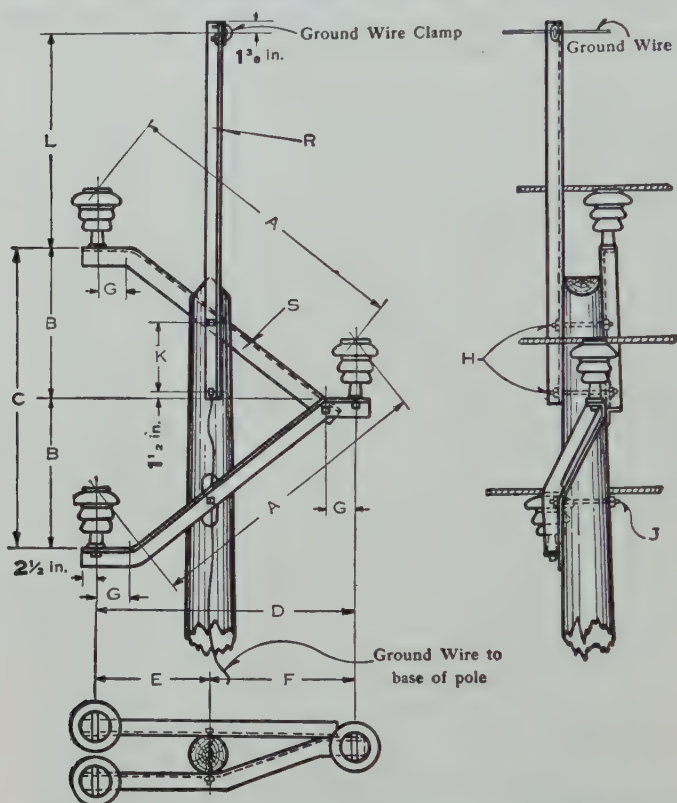


FIG. 270.—Wood pole and steel cross arm. (Courtesy of R. Thomas & Sons Co.)

No.	A	B	C	D	E	F	G	K	L	H	J
4	36 in.	18 in.	36 in.	31½ in.	13½ in.	18 in.	5 in.	12 in.	30⅝ in.	¾ x 10 in.	¾ x 10 in.
5	52 in.	26 in.	52 in.	45 in.	20 in.	25 in.	6 in.	12 in.	38⅝ in.	¾ x 10 in.	¾ x 10 in.
6	72 in.	36 in.	72 in.	62½ in.	24 in.	38½ in.	7 in.	15 in.	37⅝ in.	⅞ x 10 in.	⅞ x 10 in.

The dimensions given for Bolts are for use on 8 in. Pole Top.

R—Angles used on Nos. 4 and 5 are 2½ x 2½ x ¼ in. and for No. 6 is 3 x 3 x ¼ in.

S—Angles used on Nos. 4, 5 and 6 are 3 x 3 x ¼ in.

the construction used at angles in a 132,000-volt transmission line of 38,000-kw. capacity of the Kentucky Hydroelectric Company on the Pineville-Dix River line. This type of construction is economical in first cost and is much stronger than the single-pole or its equivalent, the flexible steel frame, although not comparable with the strength of the transmission tower, which, however, costs proportionately more. In

regions where the loading is low, as in the South, it is quite extensively used and has the advantage inherent in all wood-pole construction, that with relatively short spans (and in this case the single cross arm) the sag can be small enough to keep the whole span below the average height of trees along the right of way, thus obtaining protection from the full force of the wind, an advantage which the double-circuit standard suspension tower does not have and which, like the flexibility of the wood pole, is of considerable value. Its compactness and cylindrical shape are also advantageous in lessening wind load.

In regard to flexibility, the suspension type of insulator satisfies all requirements and is sometimes too flexible, as in the case where the wires of a circuit hang vertically over each other and a load is suddenly released in the case of sleet dropping from the middle conductor in one span only. The heavy load on the adjacent spans pulls back the wire and insulator string and possibly lets the loaded wire come into contact with the lower wire on adjacent spans or the unloaded wire to rise and come in contact with the loaded upper wire of its span. This condition is prevented by placing the middle conductor a few feet outside the plane of the other two, as in Fig. 272. On dead ends or wherever the strain-type insulator is used, and for pin-type insulators, flexibility of the structure is of importance as a factor of safety in line construction, and more consideration should be given to it in design. While a complicated problem to solve analytically, once the pull for any given tower or pole deflection is obtained, the trial-and-error method previously outlined may be used to advantage.

3. *The pole type*, with the wood pole, is shown in Figs. 269 and 270 with wood and steel cross arms, respectively; the steel pole as in Fig. 271 and the concrete pole (not shown) are classified as having a single base and are correspondingly less strong under load than either of the preceding types, as indicated by the normal span lengths of 200 to 500 ft. usual for steel poles and 100 to 350 ft. for wood poles. The noteworthy structural feature of this type is that lateral earth pressure must be relied upon for fixing the cantilever under all loading conditions, whether transverse or

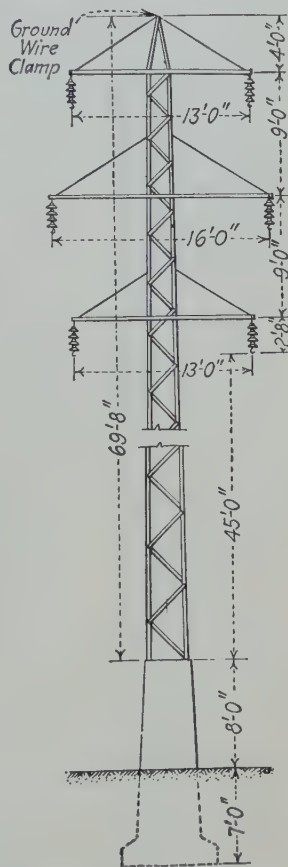


FIG. 271.—Steel pole—Turners Falls Power and Electric Company.

in the direction of the line, while, with the wood H-frame, reliance need be placed on lateral resistance in one direction only, and for towers the cantilever is held by weight alone.

Steel poles are used in populated districts, rather than for important cross-country lines, because of the cost and difficulty of procuring a right of way for large-based towers. Steel towers are also used for light low-voltage lines. Both the pin-type and suspension insulators are used for this type of structure, and one or two circuits are carried, although it is better structurally to use but one circuit with pin-type insulator for standard line construction, on account of the difficulty and uncertainty of providing sufficient lateral restraint in the foundations. This type of tower, however, carrying two heavy circuits, as in Fig. 271 is required in special cases, for right-of-way reasons, as mentioned above, although from a structural point of view it is more costly and less reliable than the wide-base tower. Furthermore, a concrete foundation is always required for the steel pole, and to a depth in the ground consistent with Table 98, for wood poles of the same height.

TABLE 98.—POLE SETTINGS
(Committee on Overhead Line Construction, Nat. Electric Light Assoc.)

Length of pole overall, feet	Recommended minimum depth in ground, feet	
	Straight lines	Curves, corners, and points of extra strain
30	5.0	6.0
35	5.5	6.0
40	6.0	6.5
45	6.5	7.0
50	6.5	7.0
55	7.0	7.5
60	7.0	7.5
65	7.5	8.0
70	7.5	8.0
75	8.0	8.5
80	8.0	8.5

The design of steel poles presents no new problems in structural design, although the torsion produced by unbalanced loading, particularly if suspension insulators with their wide horizontal conductor spacing are used, may add considerably to the stresses all the way down the pole, while the greatest effect of torsion is limited to the top of the tower.

With wood poles, the best practice is to use the pin insulator, as, with the comparatively short spans which are allowable, the conductors themselves when rigidly attached to the pole serve as an additional support for it. One circuit only should be used with wood-pole lines, for the reasons previously stated. There is no difference in the structural

design of wood poles from that of other wooden structures. The weakest section of a wood pole in bending, as may be shown, is at the point where the diameter is one and a half times the diameter at the point where the load is applied. This may occur, depending on the taper of the pole, above, below, or at the ground line. In soft ground special anchorage must be provided, such as a timber bolted horizontally to the pole a few feet below the ground line and sometimes at the base of the pole as well, to increase the lateral resistance of the earth. In ordinary earth the pole settings given in Table 98 should be used. A study of the safe lateral amount of earth pressure which will be of value in designing foundations of steel poles can be made from this table by equating the safe or working strength of the average pole of the given height in bending against the depth of embedment given. In the computation of resistance, at least the first foot of earth should be neglected. For detailed specifications on wood-pole construction, those of the committee on Overhead Line Construction of the National Electric Light Association are recommended, as well as their detailed specification for the various kinds of wood used for poles and cross arms, and the general specifications as to quality which apply to all poles and are given briefly here.

TABLE 99.—CLASS A POLE DIMENSIONS
(Committee on Overhead Line Construction, Nat. Electric Light Assoc.)

Length of pole, feet	Circumference, inches							
	Chestnut		Eastern white cedar		Western white cedar		Creosoted southern yellow pine ¹	
	Top	6 ft. from butt	Top	6 ft. from butt	Top	6 ft. from butt	Top	6 ft. from butt
20	..	..	..	..	28	30		
25	..	..	..	..	28	34	22	33
30	24	40	24	40	28	37	22	35
35	24	43	24	43	28	40	22	38
40	24	45	24	47	28	43	22	40
45	24	48	24	50	28	45	22	42½
50	24	51	24	53	28	47	22	44½
55	22	54	24	56	28	49	22	47
60	22	57	24	59	28	52	22	49
65	22	60	..	..	28	54	22	51
70	22	63	..	..	..	..	22	53
75	22	66	..	..	..	..	22	55
80	22	70	..	..	..	..	22	57
85	22	73						
90	22	76						

¹ Long-leaf, short-leaf, or loblolly.

Poles must be reasonably straight, of not less than a certain diameter at the top, and 6 ft. above the butt, as given in Table 99. Dead wood, green, fire-killed wood, large knots, cracks, twists, wind shakes or seasoning checks, sap rot, butt rot, and cat faces are the defects considered in fixing the class or quality of the pole. There are four classes, only two of which are used for transmission purposes, class A, commonly used, and class B for light lines only.

The woods generally used for poles are listed as follows in their common order: northern or white cedar, chestnut, western or red cedar, and several varieties of hard pine, although this by no means completes the list, as many other woods are used locally. Chestnut, however, due to destruction by blight, is no longer obtainable in quantity in most sections of the country, and the use of pine is increasing.

Cedar, and chestnut to a lesser degree, can be used without preservative treatment, while pine cannot. It is probable, however, that butt treatment of all poles will add enough to the life to warrant the extra expense. There are three methods of treatment, given in order of their relative protection which is proportional to the weight of preservative absorbed, the pressure treatment, the open-tank process, and the brush treatment. Creosotes and other tar derivatives are used for the preservative.

The values of ultimate strength, etc., of timber are given in Table 100. A factor of safety of from 2 to 6 is used, the higher factor for loadings with

TABLE 100.—STRUCTURAL PROPERTIES OF WOOD, POUNDS PER SQUARE INCH

	Modulus of rupture	Modulus of elasticity	Fiber stress at elastic limit
Long-leaf pine.....	8700	1,630,000	5400
Loblolly pine.....	7500	1,380,000	4400
Short-leaf pine.....	8000	1,450,000	4500
Chestnut.....	5600	930,000	3100
Western red cedar.....	5200	950,000	3300
Eastern white cedar.....	4200	640,000	2600

From "Mechanical Properties of Woods Grown in the United States," U. S. Dept. Agr., *Bull.* 556.

no allowance for broken-wire conditions, and the small factor where full allowance is made.

Classification by Loading Requirements.—Having classified the types of transmission structures in accordance with their relative strength, a further mechanical classification consists in grouping the structures of any type in accordance with the severity of loading for which they are designed. The conductor, as stated, has three load components: (I) the vertical component or weight of the wire and ice; (II) the horizontal

component transverse to the line or wind load on the wire; and (III) the horizontal component of the pull in the wire in the direction of the line. Additional forces acting are (IV) the wind on the structure and (V) the weight of the structure. It is common practice to design for the full transverse loading, II and IV, and the vertical loads I and V, and to provide only for a part of load III, as the wire pull on tangents is ordinarily balanced.

To provide for the unbalanced condition which results when one or more conductors are broken, a certain percentage of load III is usually assumed as acting in the direction of the line. The percentage allowed is the basis for the classification of transmission structures into (1) standard line structures, where the percentage varies from 5 to 67 per cent with 33 to 40 per cent representing best practice; (2) the semistrain and angle structures, with a percentage varying from 25 to 100 per cent, with no standard practice, as the uses of these types are too varied to classify; and (3) the dead-end type (Fig. 259) which is designed for 100 per cent, or the condition where all conductors are assumed broken in one span and none in the adjacent one.

The increase in strength for the semistrain or angle tower and the dead end is usually obtained in towers by increasing the strength of the towers and the weight of the anchorage, and occasionally by guying, as in Fig. 258, while in A- and H-frame and pole-line construction it may be obtained either by guying, as in Fig. 268, or by substituting towers at the desired intervals. The semistrain type is used at angles in the line from 5 to 30 deg. and at places in the line where a fairly heavy structure is required, while dead ends are necessary wherever the tension in the line is greatly changed, as in long-span river crossings or for railroad crossings where dead ends are required.

Classification Electrically.—Transmission structures can be classified electrically into two distinct types which have been fairly well standardized by practice, the single-circuit tower as shown in Fig. 254 and the double-circuit tower as shown in Figs. 272 and 273. The single circuit, on account of its relatively greater strength and lower height due to horizontal spacing, is adapted to large-capacity long-span high-voltage construction, and, while used largely for single circuits, it may also be used for double circuits at a lower voltage, which allows a closer spacing of conductors, as was done in the case of the 110,000- to 220,000-volt Davis Bridge-Millbury line of the New England Power Company.

A double-circuit tower is adapted to lower voltages where the quantity of power transmitted in a single circuit is so much reduced as to make the single-circuit tower too costly. In addition, under the electrical classification there are several special types of towers, such as the cross-over used where high-voltage lines cross one another; the switching tower, a single-circuit type which brings the wires on a double-circuit line from a

vertical to a horizontal plane at the towers; and the transposition tower, which is used to change the relative plane of the conductors.

Spacing of Conductors and Clearances. *Ground Clearance.*—The conductor under average specifications which vary with the voltage must,

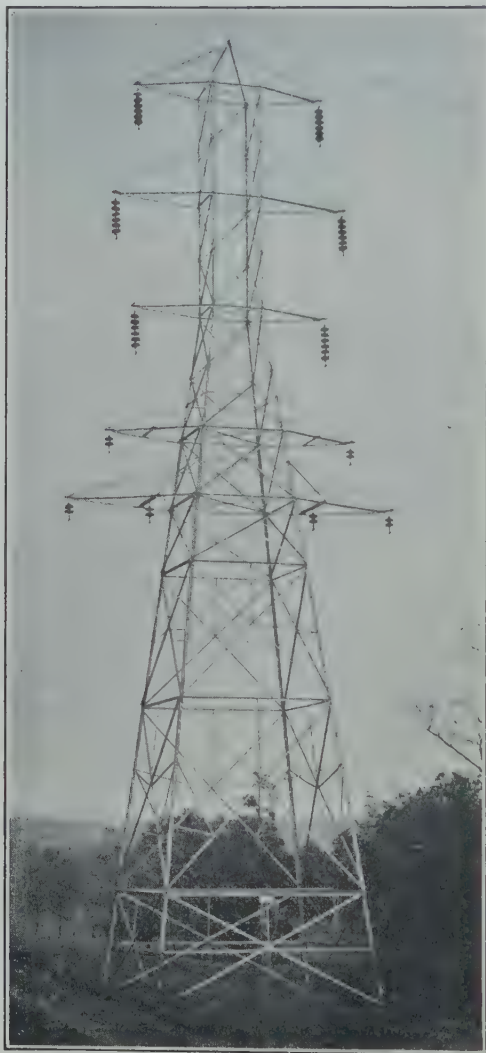
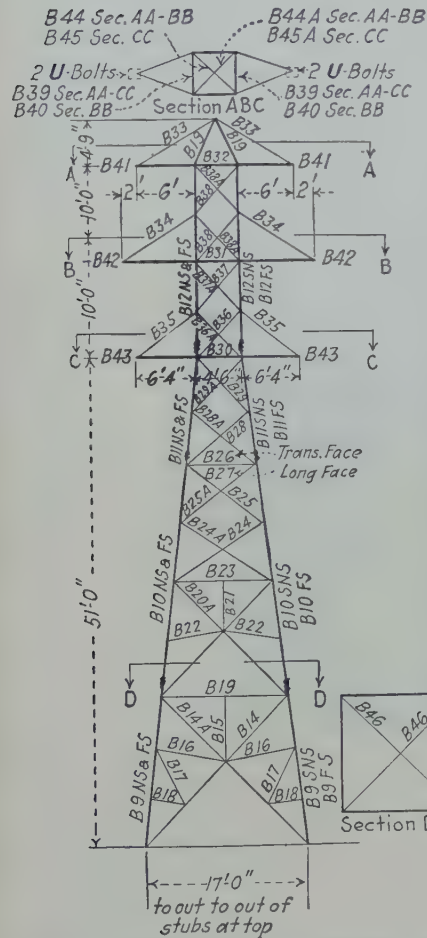


FIG. 272.—Tower for 66,000-volt Pittsfield line of Turners Falls Power and Electric Company with two 13,000-volt circuits (for Greenfield) and 10-ft. tower extension.

under conditions of maximum sag, clear the ground by 20 and 35 ft. over highways. Tower lines are ordinarily designed to clear about 25 ft. This requirement fixes the height of the lowest wire at the supports for any given sag and span at about 45 ft. for an average span of 500 to 600 ft. and is the most important requirement, other than the size of the con-

ductor, in affecting the cost of the supporting structures. Where navigable rivers have to be crossed, large clearances are required, from 50 to 150 ft., and the cost increases greatly, particularly where the structure is dead-ended and the span is 1000 ft. or over.



Mark	No.	Material	Weight, lb.
B9	3	3 × 3 × 1/4 × 17'-9 1/16"	262
B9S	1	3 × 3 × 1/4 × 17'-9 1/16"	87
B10	3	3 × 3 × 1/4 × 24'-9 1/16"	353
B10S	1	3 × 3 × 1/4 × 24'-9 1/16"	117
B11	3	3 × 3 × 1/4 × 11'-43 1/8"	140
B11S	1	3 × 3 × 1/4 × 11'-43 1/8"	47
B12	3	3 × 3 × 1/4 × 19'-73 1/8"	122
B12S	1	3 × 3 × 1/4 × 19'-73 1/8"	41
B13	4	3 × 2 × 1/2 × 5'-9"	38
B14	4	3 × 3 × 1/4 × 21'-05 1/8"	107
B14A	4	3 × 3 × 1/4 × 21'-05 1/8"	107
B15	4	3 × 3 × 1/4 × 7'-23 1/8"	36
B16	8	3 × 3 × 1/4 × 7'-23 1/8"	71
B17	8	3 × 3 × 1/4 × 6'-03 1/8"	67
B18	8	3 × 3 × 1/4 × 3'-83 1/8"	30
B19	4	3 × 3 × 1/4 × 12'-11 7/8"	108
B20	4	3 × 3 × 1/4 × 16'-2 1/8"	80
B20A	4	3 × 3 × 1/4 × 16'-2 1/8"	80
B21	4	3 × 3 × 1/4 × 5'-73 1/8"	21
B22	8	3 × 3 × 1/4 × 5'-73 1/8"	45
B23	4	3 × 3 × 1/4 × 10'-103 1/8"	84
B24	4	3 × 3 × 1/4 × 10'-103 1/8"	54
B24A	4	3 × 3 × 1/4 × 10'-103 1/8"	54
B25	4	3 × 3 × 1/4 × 9'-81 1/8"	48
B25A	4	3 × 3 × 1/4 × 9'-81 1/8"	48
B26	2	3 × 3 × 1/4 × 7'-3 1/8"	57
B27	2	3 × 3 × 1/4 × 7'-3 1/8"	29
B28	4	3 × 3 × 1/4 × 8'-39 1/8"	41
B28A	4	3 × 3 × 1/4 × 8'-39 1/8"	41
B29	4	3 × 3 × 1/4 × 7'-4 1/8"	36
B29A	4	3 × 3 × 1/4 × 7'-4 1/8"	36
B30	2	3 × 3 × 1/4 × 17'-73 1/8"	73
B31	2	3 × 3 × 1/4 × 20'-95 1/8"	128
B32	2	3 × 3 × 1/4 × 7'-0"	70
B33	4	Bars 3/4 × 3/16 × 9'-73 1/8"	31
B34	4	Bars 3/4 × 3/16 × 10'-15 1/8"	32
B35	4	Bars 3/4 × 3/16 × 8'-95 1/8"	28
B36	4	3 × 3 × 1/4 × 6'-67 1/8"	32
B37	4	3 × 3 × 1/4 × 6'-67 1/8"	32
B37A	4	3 × 3 × 1/4 × 6'-67 1/8"	32
B38	8	3 × 3 × 1/4 × 6'-69 1/8"	53
B38A	8	3 × 3 × 1/4 × 6'-69 1/8"	53
B39	4	3 × 3 × 1/4 × 4'-45 1/8"	18
B40	2	3 × 2 × 1/2 × 4'-45 1/8"	14
B41	2	6" × 8.25 9"	12
B42	2	6" × 8.25 9"	12
B43	2	6" × 8.25 9"	12
B44	2	3 × 3 × 1/4 × 6'-11 1/8"	12
B44A	2	3 × 3 × 1/4 × 6'-11 1/8"	12
B45	1	3 × 3 × 1/4 × 6'-134 1/8"	6
B45A	1	3 × 3 × 1/4 × 6'-134 1/8"	6
B46	2	Bars 1/4 × 3/16 × 18'-31 1/8"	29
Bolts, washers, etc.			141
Stubs			3,357
Total weight of tower			521
			3,878

FIG. 273.—Standard Tower—Pittsfield Line, Turners Falls Power and Electric Company.

Tower Clearance.—The clearance of the conductor from the tower is based on the sparkover distance, which increases with the voltage, as indicated in Table A of the general specifications of the American Bridge Company for Steel Transmission Towers and Poles, which follows. This gives the clearance for pin-type insulators and for suspension insulators for two cases of side swing. This requirement fixes the length of

the cross arm and the position of diagonal bracing in towers, as illustrated by Fig. 254.

*Single-circuit Semiflexible Type of Tower.*¹—This type of line, known as the “New England power design,” sponsored by the New England Power Association, is somewhat novel in practice.

In this design the line is considered to be divided into a number of structural units, each about $\frac{3}{4}$ mile in length, and consists of one anchor

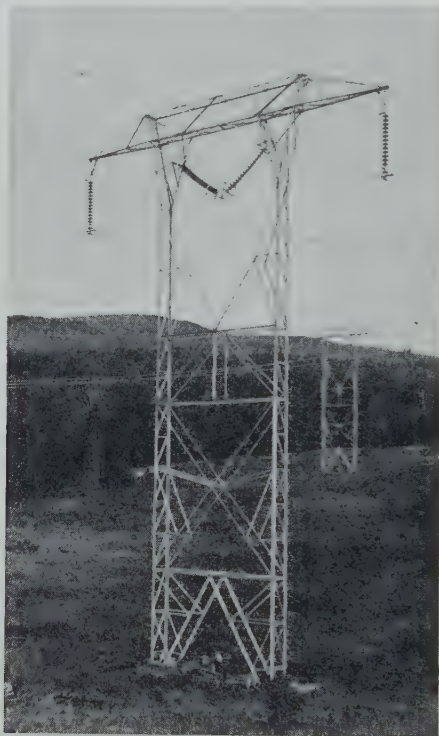


FIG. 274.—Fifteen Mile Falls transmission line, semiflexible type of towers, two 220-kv. circuits. (Courtesy of New England Power Association.)

tower at each end and one or more intermediate flexible towers, all of which are securely tied together with two high-strength steel ground wires attached to the top of the legs.

Each flexible tower (Fig. 274) is designed to carry as an independent unit all transverse loads from wind forces but to be flexible in the direction of the line, so that in case of unbalanced longitudinal loads it will deflect away from the break until the loads are again balanced by unbroken wires. Thus the longitudinal load is transferred to the high-strength ground wire, which in turn transmits it back to the nearest anchor tower, where it is carried into the foundations. It is only necessary to provide

¹ BOOKER and WEBER, “Transmission Line Structures,” *Jour. Boston Soc. Civil Eng.*, February, 1930, p. 43.

footings for longitudinal load resistance at the strain tower locations, averaging about $\frac{3}{4}$ mile apart.

The flexible tower is designed with four legs in pairs, each pair being close enough together so that a single footing can be used for both legs. Tests have shown that the flexibility of such a tower occurs at or below the ground line, so that earth anchors are always used.

This type of construction gives two independent lines at practically the cost of a single double-circuit line and permits a low average height of conductor owing to the greater number of towers per mile, thus lessening the possibility of induced lightning disturbances.

The line of the New England Power Association from the Fifteen Mile Falls plant on the upper Connecticut River to Tewksbury, Mass., substation, two-220-kilovolt circuits, is of the semiflexible, double-circuit type. The conductors are 795,000 cir. mils, A.C.S.R., 1.09 in. in diameter, ultimate strength 27,000 lb. per square inch. The ground wires are $\frac{1}{2}$ -in. plow steel, ultimate strength 180,000 lb. per square inch, with strength and other characteristics comparable to the conductors. An ice load of $2\frac{1}{2}$ -in. radial thickness can be carried with every seventh tower of the strain type.

The towers are 53 ft. high to the cross arm, which is 47 ft. long, with the circuits 183 ft. apart on centers and a 350-ft. right of way. In the 130 miles to Tewksbury, 1930 semiflexible towers are used and 420 square towers. This makes the average spacing of towers about 587 ft. and the strain towers about 4100 ft. apart.

GENERAL SPECIFICATIONS FOR STEEL TRANSMISSION TOWERS AND POLES

DESIGN

Kind of Material.—The material for the superstructure shall be structural steel, except for special fittings and as may be otherwise specified. The material for the foundations may be either steel or concrete, or a combination of both.

Outline.—The outline of the tower shall be such as will provide sufficient clearance between conductors and tower members and between all wires or cables. The structure shall be made so that the stresses may be computed with reasonable accuracy.

Height.—The height of tower shall be such as will give proper overhead clearance of lowest conductor and shall be determined by the kind of conductor used, span length, climatic conditions, and length of insulator.

Base.—The dimensions of the base of tower shall be such as will give the most economical structure, right-of-way conditions being considered.

Clearances.—The clearance between conductors and the nearest tower member shall be not less than that given in inches in Table A.

Case 1—Pin insulators.

Case 2—Suspension insulators swung 45 deg. from vertical.

Case 3—Suspension insulators swung 60 deg. from vertical (due to wind load).

If due to angle in line, use case 2.

Spacing of Conductors.—The spacing of conductors varies with the voltage, span length, kind of conductors, insulators used, and the relative position of conductors. The spacing shall be not less than that given in Table B.

TABLE A.—CLEARANCE BETWEEN CONDUCTORS AND NEAREST TOWER MEMBER

Volts	Case 1	Case 2	Case 3
2,200	12	12	11
4,400	12	12	11
6,600	12	12	11
11,000	12	12	11
13,200	12	12	11
16,500	12	12	11
22,000	12	12	11
33,000	15¼	15¼	13½
44,000	19	19	17½
55,000		23	20½
66,000		26½	24
88,000		34¼	31
110,000		42	38
140,000		52½	47¼
150,000		56	50½
165,000	...	61	55

TABLE B.—MINIMUM SEPARATION OF CONDUCTORS
(Distances given in inches)

Pin insulators						
Kilovolts	Span length in feet					
	150	250	350	500	650	800
Not exceeding 6.6.....	24	30	36	48	60	72
Over 6.6 to 22.0.....	32	37	43	54	65	76
Over 22.0 to 44.0.....	44	49	54	63	73	82
Over 44.0 to 66.0.....	58	62	66	74	82	89
Over 66.0 to 88.0.....	72	75	78	84	90	96
Suspension insulators						
Kilovolts	Span length in feet					
	150	250	350	500	650	800
Not exceeding 44.0.....	48	56	62	72	86	100
Over 44.0 to 66.0....	66	72	79	88	101	114
Over 66.0 to 88.0....	79	85	91	101	112	124
Over 88.0 to 110.0....	92	98	103	112	123	134
Over 110.0 to 140.0....	104	110	114	124	134	144
Horizontal offset of middle conductor for vertical spacing of circuit.....	12	15	24	24	24	30

Where a ground wire is necessary, its location shall be such that under condition of severest loading there can be no interference, either electrical or mechanical, with the conductors.

The distance between the ground wire and the nearest conductor, at their points of support, shall not be less than 70 per cent of that allowed between conductors, assuming the ground wire to be made of steel and the conductors of copper or aluminum. If the ground wire is made of the same material as the conductors, the same minimum clearance shall apply as for conductors.

Loads.—Towers shall be designed for the following assumed loads:

1. Wind on towers and cables.
2. Pull of cables.
3. Weight of towers, insulators, cables, and ice.

1. The wind load on the tower shall be not less than 13 lb. per square foot, acting in a horizontal direction, on one and one-half times the exposed area of one face; and that on cables, covered with a coating of ice $\frac{1}{2}$ in. thick (= diameter of cable + 1 in.), 8 lb. per square foot of projected area acting horizontally at right angles to directions of line.

NOTE: When towers are to be used in a country where ice does not form, the ice load may be omitted, in which case the wind load on the tower shall be not less than 25 lb. per square foot, on one and one-half times the exposed area of one face; and that on the cables not less than 15 lb. per square foot of projected area of cable.

For towers in a country where heavy sleet is common, also for lines of exceptional importance, class C loading or 11 lb. per square foot of projected area of cable covered with a coating of ice $\frac{3}{4}$ in. thick (= diameter of cable + $1\frac{1}{2}$ in.) shall be used.

2. The maximum pull from a cable will be determined by the engineer and may be such as will stress the cable to its yield point, but a less stress is usually advisable. The pull will be applied horizontally in the direction of the line at its point of support on tower.

3. The weight of tower, insulators, and cables shall be calculated, the cables being covered with a coating of ice $\frac{1}{2}$ in. thick, if such a condition is liable to occur in the district where the tower is located.

Types of Towers.—The types of towers to be used will be determined by the character of the line and the conditions along the right of way. At least two types are usually required, anchor and intermediate.

Anchor towers shall be used at dead ends, at angles over 5 deg. in line, at railroad and navigable river crossings.

Intermediate towers will usually be used at all points in the line where anchor towers are not used.

The general conditions obtaining along different transmission lines vary, and definite loads for which structures are to be designed should be determined for each particular line. Unless otherwise determined, the following loads shall be used.

Anchor towers shall be designed for the following loads:

1. Wind load on tower and cables, acting at right angles to the line.
2. Vertical loads due to weight of cables, insulators, and tower.
3. A horizontal load in the direction of line, at each cable support, all pulling in the same direction.
4. A horizontal load in direction of line, at each cable support on one side of tower, all pulling in the same direction.
5. A horizontal load at right angles to line, due to an angle of 45 deg. in the line, all cables being assumed as pulling.

NOTE: The following loadings shall be considered acting simultaneously: 1, 2, and 3; 1, 2 and 4; 1, 2 and 5.

Intermediate towers shall be designed for the following simultaneous loads:

1 and 2. Same as for anchor towers.

3. A horizontal load in direction of line at any one cable support.

4. A horizontal load at right angles to line, due to an angle of 5 deg. in the line, and due to all cables acting.

Unit Stresses.—All parts of the structure shall be proportioned so that the maximum unit stress will not exceed the following amounts in pounds per square inch:

For ordinary transmission towers:

Axial tension on net section.....	20,000
a. Axial compression on gross section (maximum 15,000).....	$20,000 - 85\frac{l}{r}$
b. Axial compression on gross section.....	$15,500 - 55\frac{l}{r}$
Use (a) where l/r does not exceed 150.	
Use (b) where l/r exceeds 150.	
Shear on bolts or rivets.....	13,500
Bearing on bolts or rivets.....	27,000

where l is the unsupported length of the member and r is the least radius of gyration, both in inches.

For river crossing and other important structures:

Axial tension on net section.....	18,000
a. Axial compression on gross section (maximum 14,000).....	$18,000 - 80\frac{l}{r}$
b. Axial compression on gross section.....	$13,500 - 50\frac{l}{r}$
Use (a) where l/r does not exceed 150.	
Use (b) where l/r exceeds 150.	
Shear on bolts or rivets.....	12,000
Bearing on bolts or rivets.....	24,000

DETAILS OF DESIGN

Open Sections.—The structure shall be designed so that all parts will be accessible for inspection, cleaning, and painting.

Pockets.—Pockets or depressions that can hold water shall have drain holes, or be filled with waterproof material.

Splices.—Where a lap splice is used in leg angles, the back of the inside angle shall be chamfered to clear the fillet of the outside angle.

Bolts.—The minimum size of bolts used shall be $\frac{5}{8}$ in. in diameter, except in case of very light structures.

Bolts may have rolled or cut threads but must be full size in the shank.

Bolts connecting different parts of the tower shall preferably be of the same diameter and of as few different lengths as practicable. The shank shall be long enough to extend through the members connected, washers being provided where necessary to insure tight nuts.

The necessary connection bolts including an excess of 5 per cent of the number required shall be provided with the tower material.

Bolt Holes.—Punched holes shall be not more than $\frac{1}{16}$ in. larger than the nominal diameter of bolts.

Duplication of Parts.—In the case of square towers the four faces shall be made as nearly alike as practicable.

Minimum Number of Parts.—Preference will be given to those designs having the least number of parts.

Minimum Thickness.—The minimum thickness for galvanized material shall preferably be $\frac{3}{16}$ in. For painted material the minimum thickness shall be $\frac{1}{16}$ in. more than for galvanized material.

Ratio of Slenderness.—In compression members the ratio of the unsupported length divided by its least radius of gyration shall not exceed the following:

Legs.....	140
All other members having calculated stress.....	200
Members having nominal stress only.....	250

Ground-wire Clamp.—The ground-wire clamp shall be of such design as will firmly hold but not injure the cable.

Insulator Connection.—The contractor for towers shall provide the necessary holes for connecting the insulators.

Ladder.—If desired, one leg of the tower shall be provided with steps. These steps, if made of bolts, shall be at least $\frac{5}{8}$ in. in diameter and not more than 18 in. apart on centers, starting 8 ft. from the ground.

Unless otherwise agreed upon, ladder material will not be included in contract for tower material.

Painted Work.—If the towers are to be painted, all parts of the towers so coated shall, unless otherwise specified, receive one coat of pure linseed oil at shop before shipment.

Galvanized Work.—If material is to be galvanized, all parts of the tower shall be galvanized after fabrication by the hot-dipping method. Bolts and other special parts shall be either sherardized or, preferably, hot-dip galvanized. If bolts are galvanized, the threads of the nuts may be uncoated. Threads of nuts and bolts shall be made so that, after galvanizing or sherardizing, they will have a neat fit and the nuts can be turned with the fingers throughout the length of the thread of the bolt.

All hot-dip galvanizing or sherardizing shall be done in accordance with the latest specifications of the National Electric Light Association.

Material.—Steel used shall be made by the open-hearth process and conform to the latest specifications for structural steel for buildings, as adopted by the American Society for Testing Materials.

For Canada, the specifications of the Canadian Engineering Standards Association for Railway Bridges (last edition) shall be used.

Drawings.—All parts forming a tower shall be made in accordance with approved drawings.

Workmanship.—Workmanship and finish shall be equal to the best modern practice.

Marking.—Each separate member shall be plainly stamped with a number. All like parts shall have the same number, the mark being placed in the same relative position on each piece, but to distinguish them different members shall have different numbers. This marking shall be stamped into the steel (before galvanizing or painting) in such manner as to make the marks plainly visible after galvanizing or painting.

Shipping.—Unless otherwise specified, all parts of a tower will be shipped unassembled, to be bolted together in the field. All like parts of one tower shall be bundled together before shipment, except when too heavy a bundle for convenience in handling would result.

Bolts and other small parts shall be shipped in bags, boxes, or kegs strong enough to resist rough handling.

Inspection.—On request of purchaser the manufacturer shall furnish proper facilities to the purchaser's representative for the inspection of material and workmanship during fabrication. The purchaser shall be notified well in advance of the start of work in the shop in order that he may have his inspector on hand.

Assembling.—One of each type of standard line towers shall be assembled at the shop before shipment.

Test.—At the request of the purchaser, one each of the types of standard line towers shall be assembled and tested, with loads as nearly as practicable like those for which the tower is designed. If requested by the purchaser, the tower shall be tested to destruction.

The members forming the test tower shall be selected at random from lots of similar members. The tower to be tested shall, preferably, be set on a foundation as nearly as practicable like that to be used in the field. Unless otherwise agreed upon, all tests will be made at the expense of the purchaser.

Foundations.—The foundations shall be designed to have a strength 10 per cent greater than that of the structure they support, care being taken that they are designed to resist both the horizontal and vertical loads without injurious movement.

If steel footings are used, it shall be assumed that earth weighs 90 lb. per cubic foot and that the base will engage the frustum of an inverted pyramid of earth whose sides have an angle of 30 deg. with the vertical. If concrete foundations are used, the weight of 1 cu. ft. of concrete shall be assumed to be 150 lb.

Material buried in concrete will be left uncoated except where stub angles are used, in which case the stub angles shall be galvanized (or painted, in the case of painted towers) from the top to a distance 18 in. below the top of concrete.

Conductor Clearance.—The third electrical requirement is that the conductor shall be separated to prevent contact by swinging in midspan. Here the voltage, type of insulator, and span length affect the clearances, which are given in Table B of the specifications. These may be used for the horizontal spacing, which will determine, together with the clearances given in Table A, the width of a single-circuit tower or H-frame, or the vertical spacing of cross arms as in the double-circuit tower. There are also included the proper distances to offset the middle cross arm to prevent its contact with the other conductors of the circuit under sudden changes of load, as previously discussed. For A.C.S.R. and aluminum cables, Table 101 will be of value, as it gives for class B loading conditions the proper dimensions of the structure for the different types.

Right of Way.—The distance to use in determining the width of right of way is found by adding to the position of the outside conductor, as determined by the electrical clearances given, the distance of maximum side swing under worst conditions plus a clearance from this point to the edge of the right of way, which can be taken as 15 ft. for 90,000 volts, 20 ft. for 110,000 volts, 25 ft. for 150,000 volts, and 30 ft. for 220,000 volts.¹

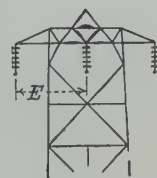
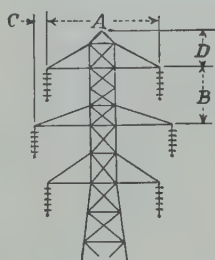
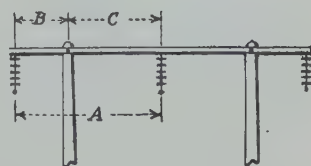
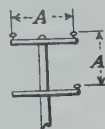
Stresses Used in Design of Steel Transmission Structures.—The live loadings on the structure and the modifications of load III for the different classes of construction have been discussed, and the clearance data by which the dimensions of the structure can be determined have been given. To design the structure it is only necessary to choose the proper unit stresses. The standard line tower is usually designed by the

¹ "Transmission Towers," American Bridge Company, p. 45.

structural steel company from an outline or dimension sketch, furnished by the engineer, giving the loading and occasionally the unit stress, but the latter is usually left to the manufacturer and the tower as designed

TABLE 101.—A.C.S.R. CONDUCTOR SPACING—CLASS B. LOADING
PIN-TYPE INSULATORS—WOOD POLES

Voltage	Size A.C.S.R. (A.w.g.)	A, inches		
		Span, feet		
		200	250	300
11,000	4	30	36	
11,000	2	30	36	42
11,000	1/0	24	30	36
11,000	4/0	24	30	30
22,000	4	48	54	
22,000	2	42	48	54
22,000	1/0	36	42	48
22,000	4/0	36	42	42
33,000	4	54	60	
33,000	2	48	54	60
33,000	1/0	42	48	54
33,000	4/0	42	48	48
44,000	4	66	72	
44,000	2	60	66	72
44,000	1/0	60	66	66
44,000	4/0	60	60	60



SUSPENSION INSULATORS

WOOD POLES

STEEL TOWERS

Voltage	Distance, feet			Voltage	Distance, feet				
	A	B	C		A	B	C	D	E
33,000	6.5	3	4	33,000	10	7	2	4	9
66,000	9.5	4	5.5	66,000	14	8	2	4	11
110,000	13.5	6	7.5	110,000	18	10	3	5	14
				150,000	23	13	4	7	17

is tested under a loading equivalent to that given and finally to destruction. It is now considered good practice to use stresses somewhat lower than formerly in transmission-line design, and those specified by the American Bridge Company and shown in Fig. 275, together with actual

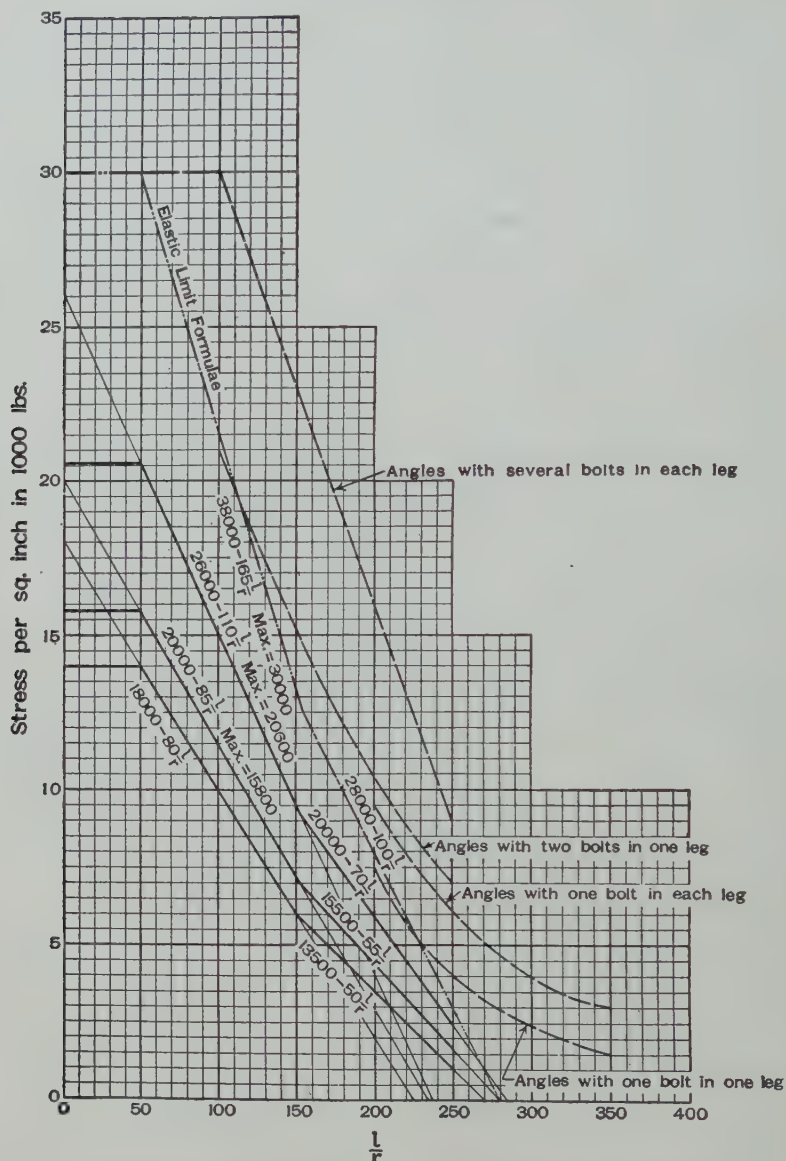


FIG. 275.—Comparison of compression formulae for steel towers. (Courtesy of American Bridge Company.)

NOTE: These formulas are developed from the results obtained from the angle tests made by the American Bridge Co. at the U. S. Arsenal, Pittsburgh, Pa., 1917. The final results for the yield points of the angles are shown as dash lines.

tests on angles with bolted connections, are among the most conservative in this respect. Other companies, however, have successfully used stresses somewhat higher for standard line towers.

The determination of the proper loading and the percentage of broken wires that should be considered for each class of structure are more important than the exact unit stress used, as the variation in allowable stresses is small compared with that of the loading. The desired result is to produce a structure that will withstand the estimated loading as erected under average conditions in the field; with a reasonable allowance for errors in stringing and overloading due to spans longer than the average; for possible higher local loading due to exposed positions of the line; uneven distribution of stress due to slight settlement of the earth anchors; the rough handling sometimes necessary in erection and delivery of the material to the site, and the possible deterioration of the metal. A factor of safety of 2 against compression failure, or about 4 against tension failure, is considered the lowest advisable. The formula of the American Bridge Company, based on actual tests, allows what is called a "possible overload" of 90 to 96 per cent, which indicates that their specifications for allowable stress for suspension or standard line towers does not give a factor of safety of quite 2. Stresses 25 per cent higher, or about the upper limit of allowable stress, would allow only about 50 per cent overload. To justify the use of such high stresses, the engineer should be satisfied that the loading conditions assumed are so liberal that they are not likely to occur. As this is generally not the case, the use of the specified stresses is usually advisable. It will be noted that for dead ends a still more conservative formula is used; this is done for the reason that, as the importance of a structure increases, its failure would involve serious problems of replacement, and the additional metal adds but slightly to the cost of the structure considering the increased reliability.

Economic Considerations.—The shape of the tower having been fixed by electrical clearances, the width of base is determined by comparing the increased cost of added weight of the web system with a wider base to the saving made possible in the weight of the legs and anchorage, as well as for excavation. Study of transmission structures indicates that the ratio of base to height should vary between 1 to 3 and 1 to 4, the former for the single-circuit and the latter for the double-circuit type, the height being taken at the intersection of the slope of the legs.

The length of span for a given conductor is also a problem of economics which consists in comparing the cost of lighter towers and a greater number of towers and insulators with that of higher, heavier, and fewer towers and a smaller number of insulators. The span is usually made as large as a reasonable sag in the conductor will allow. As the strength of the conductor is fixed, the point is soon reached where the sag, which

increases as the square of the span, will require much higher towers. The latter for a given pull increase in weight about as the square of the height, and, as the height is a function of the sag, the weight of tower increases approximately as the fourth power of the span and the limiting case is quickly reached. Thus a span of about 800 ft. with hard-drawn copper is about the maximum used for level country, while 600 ft. is more nearly in accord with average practice. Aluminum is used for shorter and A.C.S.R. for longer and "copper weld" and steel for very long spans, and, as the amount of steel in A.C.S.R. can be varied, there is no sharp limiting span for the latter material, while with aluminum the low elastic modulus and lightness restrict the length of span.

Detailed Analysis of Design.—The general method used in design is to compute the stresses for each one of the five loadings separately in determining the size of the different members of the structure. The sum of the five loadings for any member is tabulated for each of the assumed conditions, and the member is then designed for the maximum value.

With unbalanced loading or broken-wire conditions, a torsional moment or couple is exerted on the towers equal to the product of the unbalanced horizontal force and its distance to the center of the tower. The method used to compute the effect of torsion on square towers of the double-circuit type is to substitute for this couple two equivalent ones, which consist of four equal forces, each one acting in the plane of one of the faces of the tower, the value of these forces diminishing at lower heights as the resisting moment arm of the couple increases with the width of the tower. The effect of the direct pull is taken as acting equally on the two faces of the tower parallel to it in addition to the four forces of the torsion-resisting couple.

The effect of torsion on square towers under unbalanced loading is to increase the stresses in the web system only, if the web is designed as a tension and compression diagonal system, as a result of the assumption that each diagonal takes half the load, one in tension and the other in compression, as is usual in the top of the tower when the lengths of the members are short and the torsion load is most severe. That the effect on the leg is zero is obvious for the reason that in one panel the compression diagonal puts tension into the leg, while the effect of the tension diagonal from the adjacent face is to put an equal compression into the leg. In the single system used lower on the tower and sometimes for its full height, where the increased length gives a ratio of l/r over the allowable value for a compression member of reasonable size, reliance is placed on the tension member only, as the counter is too slender to have any strength in compression and the vertical component of the stress in the diagonal induced by torsion is added to the compression in the legs of the tower.

An interesting feature of design is that the inclination of the legs of the tower allows them to carry shear. If the load applied is at the point of intersection of the legs, they will carry all the figured stress, the web system acting simply as bracing for that loading. With the load applied below the point of intersection, both the diagonals and legs will help in carrying the shear, while, if the point of application is above the point of bisection, the sum of the horizontal components in the legs will be more than the total shear, and the stresses in the diagonal will be reversed.

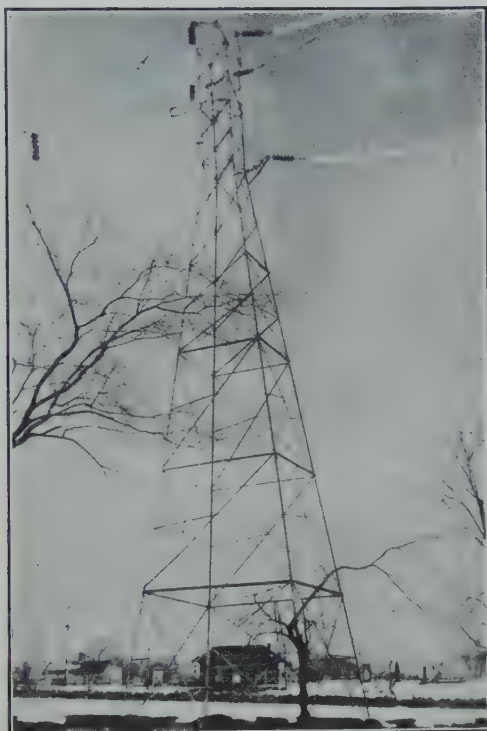


FIG. 276.—Tower failure under sleet and ice loading.

The center of gravity of the loads, therefore, should be at, and preferably a little below, the point of intersection.

Other practical considerations in the design which are important are the horizontal cross bracing, which should be designed to transfer half the shear load and to resist the equivalent forces of torsion as assumed. In addition, this cross bracing and any horizontal member should be able to support other loads which may come on it, such as the heaviest tower member or the weight of a man, as frequently required in erection, and no less shape than an angle should ever be specified for any horizontal member.

Horizontal cross bracing is indispensable at all points of concentrated loading, as no single member is perhaps as important in determining the capacity of the tower to withstand unbalanced loading.

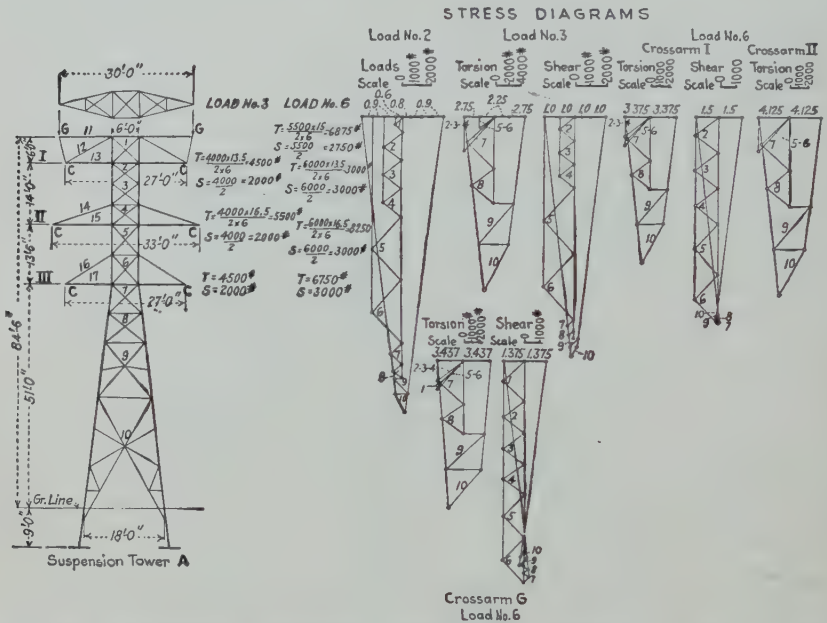


FIG. 277.—Double-circuit suspension tower. Stress diagrams. (American Bridge Company.)

Loads:

1. Vertical at G and C of 1500 #; total 12,000 #.
2. Horizontal, transverse, 1200 # at G. 1800 # at C; total 13,200 #.
3. Horizontal longitudinal, 4000 # at any two C; total 8000 #.
4. Wind on tower 40 # per square foot on $1\frac{1}{2}$ times the projected area of one face.
5. Weight of tower.
6. Horizontal, longitudinal, 6000 # at one C, or 5500 # at one G.

STRESSES IN MEMBERS IN THOUSANDS OF POUNDS

Loads	Legs				Bracing												
	4	6	9	10	1	2	3	4	5	6	7	8	9	10	11	12	13
#1	±	±	±	±	±	±	±	±	±	±	±	±	±	±	±	±	±
#2	3.0	3.0	3.0	3.0	0.9	2.0	2.0	2.0	3.7	3.7	1.3	1.0	1.9	1.8	0.7	3.0	2.7
#3T	7.3	16.8	23.8	25.5	7.8	3.0	3.9	3.0	7.7	7.7	6.3	4.8	9.2	8.7	...	1.0	2.5
#3S	4.0	11.5	15.7	16.3	...	1.3	1.3	1.3	3.0	3.0	0.6	0.4	0.8	0.7	...	7.0	...
#4	2.0	5.5	8.6	11.2	0.2	0.6	0.6	0.6	1.4	1.4	0.4	0.5	1.4	2.4	...	...	...
#5	0.5	1.5	3.0	4.0	...	...	...	...	...	...	...	...	...	...	...	...	...
Total	16.8	38.3	61.1	46.0	1.1	5.6	5.6	5.6	12.8	12.8	8.0	6.3	12.5	12.9	0.7	3.0	10.7
#6T	...	...	5.8	6.4	5.0	4.5	4.5	4.5	6.2	6.2	5.2	3.9	7.5	7.2	11.2	...	11.0
#6S	8.1	13.7	13.5	13.3	2.0	1.9	1.9	1.9	2.3	2.3	0.5	0.4	0.8	0.8	...	...	...
Combining #1 and #6	...	...	...	...	7.0	6.2	6.2	6.2	8.5	8.5	5.7	4.3	8.3	8.0	11.9	...	14.7

Stiffening the leg by short angles from the intersection of tension diagonals, which have practically no stiffness in a direction normal to their plane, should be done only with the understanding that this is a frequent cause of tower failure, as indicated by Fig. 276, and a much

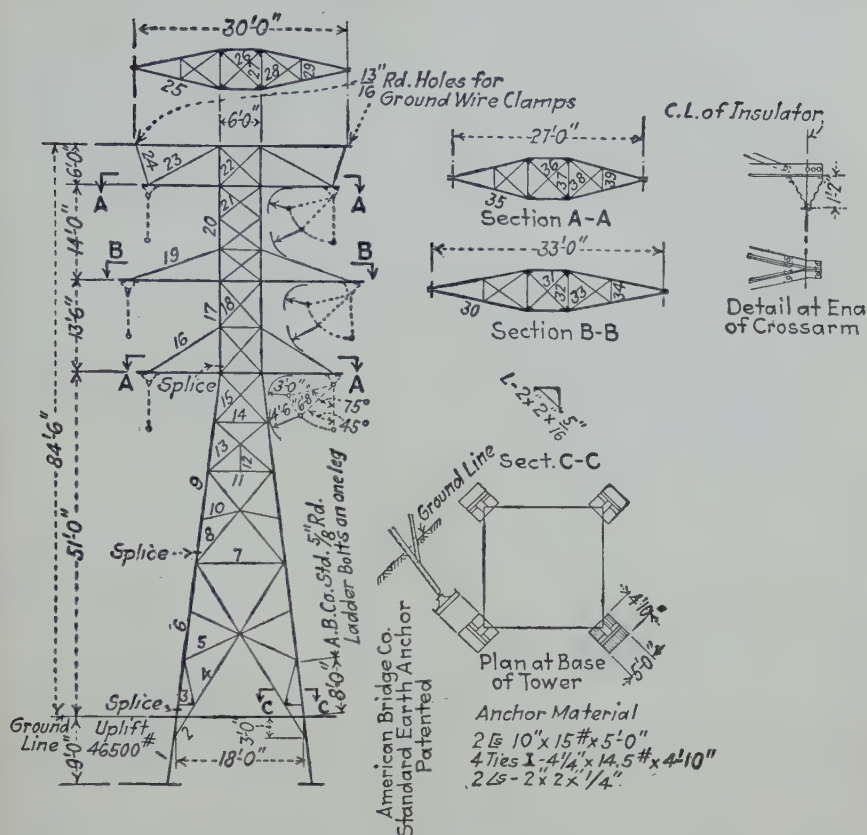


FIG. 278.—Double-circuit suspension tower design sheet. (American Bridge Company.)

Number	Stress (thousand lb.)	Dimensions of angles (or bars) (inches)
1		6 × 6 × 1/2
2		2 1/2 × 2 1/2 × 1/4
3		2 × 2 × 1/4
4	+11.6	2 × 2 × 1/4
5		2 1/2 × 2 1/2 × 3/16
6	-67.2	6 × 6 × 1/2
7		2 × 2 × 1/4
8	+12.4	2 × 2 × 1/4
9	-62.0	6 × 6 × 1/2
10		2 × 2 × 1/4
11	-5.0	2 1/2 × 2 1/2 × 1/4
12		2 × 2 × 1/4
13		2 1/2 × 2 1/2 × 1/4
14		2 × 2 × 1/4
15	+8.3	2 1/2 × 2 1/2 × 1/4
16	+1.5 ea.	2 Bars - 1 1/4 × 1/16
17	-38.1	5 × 5 × 3/8
18	+12.7	3 × 3 × 1/4
19	+2.5 ea.	2 Bars - 1 3/4 × 3/16
20	-16.5	3 1/2 × 3 1/2 × 1/4

Number	Stress (thousand lb.)	Dimensions of angles (or bars) (inches)
21	± 6.2	2 1/2 × 2 1/2 × 3/16
22	± 5.6	2 1/2 × 2 1/2 × 3/16
23	+3.0 ea.	2 Bars - 1 1/2 × 1/4
24		2 × 2 × 1/4
25	-12.9	4 × 3 × 1/4
26	+6.5	Bar 2 1/2 × 1/4
27		2 1/2 × 2 1/2 × 3/16
28		Bar 1 3/4 × 3/16
29		2 × 2 × 1/4
30	-17.0	5 × 3 × 3/8
31	+7.5	Bar 2 1/2 × 1/4
32		2 1/2 × 2 1/2 × 3/16
33		Bar 1 3/4 × 3/16
34		2 × 2 × 1/4
35	-15.9	4 × 3 × 3/16
36	-5.5	Bar 2 1/4 × 1/4
37		2 1/2 × 2 1/2 × 3/16
38		Bar 1 3/4 × 3/16
39		2 × 2 × 3/16

DATA

The tower shown on this sheet is designed to support the bare cables with a wind load on flat surfaces of 40 lb. per square foot, the corresponding pressure on cables being 25 lb. per square foot. The temperature range is from -4.0° to $+122.0^{\circ}$ F.

It is also designed to support the cables covered with 1/4-in. shell of ice, with a wind load of 20 lb. per square foot on flat surfaces or $12 1/2$ lb. per square foot on the ice-covered cables, the temperature range being -4.0° to $+32.0^{\circ}$ F.

(Continued on page 632)

larger value should be used for the unsupported length than the exact dimension, or the stiffening member may be carried across to the opposite leg and figured for a ratio of l/r for more nearly its full length, or the intersection may be cross braced, as is sometimes done.

The design sheets are given in Figs. 277 and 278 for a typical suspension or standard line tower as designed by the American Bridge Company in accordance with their specifications, which are also included as a complete example of a well-standardized method of design which is conservative throughout and, therefore, safe to follow. This refers to the specifications rather than the tower designed, which is for a severe loading and for 154,000-volt transmission. As stated previously, the greater the power transmitted, the more liberally the loads should be estimated, particularly with two circuits, as the value of reliability far exceeds the slight added cost of steel necessary for conservative design.

Foundations.—Foundation design is ordinarily a problem of providing sufficient weight of earth over an anchorage connected to the tower leg to restrain the uplift due to the cantilever action of the forces acting on the tower. The anchorage should be embedded in the earth 6 ft. or more, depending on the amount of uplift, and, if properly designed for uplift, it will usually be adequate for maximum allowable compression. Here again the limiting factor of safety of 2, at least, should be applied to the loads delivered to the foundation to make it as strong as, and preferably slightly stronger than, the tower and to prevent any movement under normal loads. The specifications, previously referred to, give a method of calculating the weight of the anchorage. This is based on tests which indicate that the actual weight lifted has nearly vertical sides, the remaining resistance being derived from friction. The weight is for ordinary earth; it will be greater for soils of greater cohesion like clays and less for sand having little or no cohesion.

It is assumed that the cables will be so strung that under the worst conditions the maximum tension in same will be 6000 lb. in the conductors and 5500 lb. in the ground wires.

The various loads are as follows and are based on a span of 650 ft. with a 5-deg. angle in the line. All loads to be combined.

1. A vertical load at each of the eight cable supports of 1500 lb.—total 12,000 lb.
2. A horizontal load transverse to line of 1200 lb. from each of the two ground wires and 1800 lb. from each of the six conductors—total 13,200 lb.
3. A horizontal pull in the direction of the line of 4000 lb. from each of any two conductors—total 8000 lb.
4. Wind on tower of 40 lb. per square foot of one and one-half times the projected area of one face of the tower.
5. Dead load of tower.

In addition to the above loads the tower is also designed to be subjected to a single load from any one conductor of 6000 lb. and from any ground wire of 5500 lb.

Specifications:

Unit stresses:

Tension on net section = 20,000 lb. per square inch.

Compression on gross section = $20,000 - 85 \frac{l}{r}$

Shear on bolts = 13,500 lb. per square inch.

Bearing on bolts = 27,000 lb. per square inch.

Material: O. H. steel for buildings. A.S.T.M. standard specifications.

Coating: All structural material to be galvanized.

Connections: Bolted. Bolts to be checked in field.

Balance of specifications—American Bridge Company's standard for transmission towers, which also embodies the above notes.

Tests also show that a pressure of 60 to 80 lb. per square inch causes the anchor to pull through the earth.

Figures 279 and 280 show several types of foundations which are commonly used for transmission towers. Concrete anchors (Fig. 279)

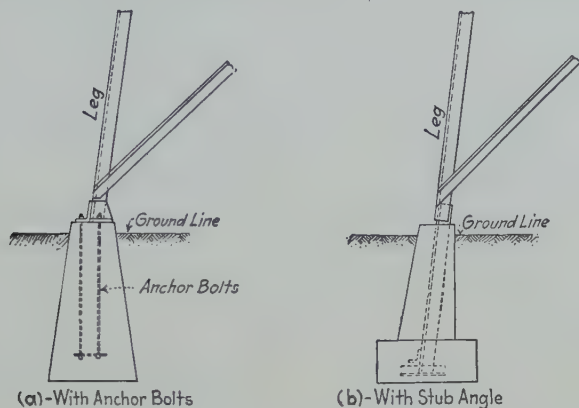


FIG. 279.—Concrete anchors—American Bridge Company.

are seldom used for standard line transmission, except in soft ground where lateral resistance is required at the ground line and little resistance to uplift would be provided with an ordinary steel anchor (Fig. 280). The type *a* is now standard with the American Bridge Company and differs from the usual type *b* in that it brings the lowest panel point of

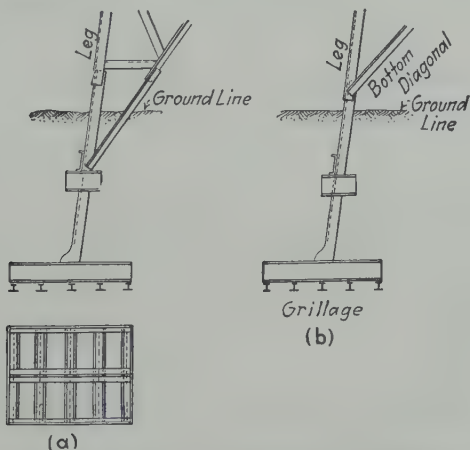


FIG. 280.—All-steel anchors for earth setting—American Bridge Company.

the tower far enough below ground to insure sufficient earth cover to resist the horizontal component of the pull in the diagonal, which otherwise will cause bending in the tower leg at the panel point. Many towers are built in accordance with type *b*, usually without the channel

at mid-depth and with a smaller grillage, which indicates that earth in general must exert considerable lateral resistance.

For dead ends and river crossings, the foundations become heavy and costly, particularly for the latter, as they may be submerged and are sometimes subject to ice pressure, which greatly increases the necessary lateral resistance. In such large structures the effect of lateral resistance of the earth is frequently neglected; in fact, it is common practice to do so.

Choice of Type of Supporting Structure.—The question of the choice of structure is also a subject on which there is much difference of opinion. Some companies use wood-pole lines for important power transmission, other companies use steel-tower lines for lines of relatively small capacity. There are three things to be considered: (1) the cost of the service rendered, (2) the character of the service rendered, and (3) the results of interruption to service. For example, if a transmission company owns two or more lines by which a given market can be served, it would unquestionably be inadvisable to spend as much money on each circuit as it would if it were an isolated market connected only by one transmission line. For primary transmission purposes, the transmission structure, whether of wood or steel, should be built under rigid specifications for strength, for the loading that will probably come on the structure, as interruptions to service from failure would be much more expensive than the small additional cost of making the structure safe to withstand the worst conditions.

ECONOMIC CONSIDERATIONS INVOLVING THE CHOICE OF CONDUCTOR

The voltage of a transmission line depends on the distance of transmission. A common rule is to make the kilovolts equal the transmission distance in miles where no other factors enter into the decision. This will be found to be a satisfactory rule, as the principal voltages which are standard do not offer a very great field of choice for any particular installation. It will be frequently found, however, that from an operating standpoint there is a considerable advantage in adhering to the voltage used in the adjoining circuits, although it may not be so well adapted to the line in question.

The rule of distance equal to kilovolts has several exceptions: first, where the amount of power to be transmitted is very small, it may be found less expensive to use a lower voltage at the expense of a large line loss; second, where the amount of power is so great that it can be handled more economically by using a higher voltage for the purpose of decreasing the number of circuits or tower lines. In all cases the decision as to a proper voltage depends on the length of the line, the amount of power, the usual voltage of adjacent systems, and the anticipated growth in

load. Where a choice between two voltages is possible, a study of the economies of both should be made.

Assuming that the voltage has been chosen, the next step is to determine the size of the conductor. As previously stated, the cost of a line for any given type of construction and type of conductor material is fairly rigidly fixed by the size of the conductor, as almost all of the items entering into the construction cost of the line are determined by the working stress in the conductor.

There are three types of conductors in use today for standard line transmission: first, copper; second, aluminum; third, aluminum cable steel reinforced. The use of aluminum is not great and is limited to relatively short spans where the loading is not excessive. Copper wire and steel-reinforced aluminum are the two which are considered for ordinary transmission work. The former has its field in short- to medium-length spans, while the aluminum steel-reinforced cable is perhaps better adapted to extreme high-tension transmission involving medium to long spans. Special cables are used for very long spans, such as copper weld, plow steel, etc.

A uniform structural classification of conductor, regardless of the material, size, or stranding, is possible by determining for each conductor the allowable pull T based on 90 per cent of the elastic limit and dividing unit pull by W , the resultant load per foot at the loading considered. This value T/W is the working stress in the conductor for a resultant load of unity and is inversely proportional to the sag and, if plotted against effective alternating-current resistance, will serve as a useful guide in selecting a wire that will have desirable electrical and structural qualities.

In steel-tower transmission the cost of the conductor is from 10 to 15 per cent of the total cost of the line and on special long-span construction reaches as low a figure as 3 per cent. The author is convinced that there is need and demand for improvement in conductors from a structural point of view, as much more money could be justifiably spent on the conductor if the desirable structural qualities could be obtained, as the total cost for any type depends almost entirely on its selection. Too little attention has been paid to the choice of conductor from this standpoint, particularly on long-span construction, where the electrical requirements are greatly outweighed.

Assuming that the problem is one in which the type of conductor can be readily decided, it is evident that for a given line loss a conductor of certain definite size must be used requiring a definite yearly amount for fixed charges, operating, and maintenance. If the annual value of the power lost by this conductor is added to this annual cost, a total annual cost of transmission will be determined. If another conductor somewhat different in size is chosen and the yearly value of lost power and fixed

charges determined, it will be found that their sum is greater (or less, depending on the second assumed size) than that obtained for the first conductor. Further examination of the next-sized conductor (by the use of a plot) will usually indicate the desirable size for the circuit. This is an exact method of determining the most economical size of conductor, assuming that neither is the current excessive nor is corona present, as sometimes may be the case. It is analogous in principle to the method described in Chap. VI, page 336, for the determination of best size of canal or penstock.

In determining the power lost, the load used should not be the present expected load but rather the average expected load on the transmission line during its life. As it is frequently impossible to determine exactly the amount of load which will be carried during the average life of a transmission line, as well as many other factors necessary for the economic study, many engineers determine the proper size of conductor by allowing a definite per cent loss in the line. This percentage, based on experience, varies with the amount and quality of power to be transmitted, the distance and number of circuits or lines which are available to supply the load in question, and in general the nature of the service to be rendered. Regulation is also an important factor from an electrical standpoint, and a conductor which might be satisfactory from an economical standpoint might not be large enough to keep the voltage variation within proper limits.

It is a difficult matter to determine completely and accurately the actual construction cost for three different sizes of conductor, and often time is not available for accurate estimates. In the usual case the size of conductor is determined by the line-loss method and an accurate cost study made. Then the next larger and smaller sizes may be examined to show whether any saving is possible.

It is sometimes possible to effect a saving by using a different conductor in one portion of a line where special conditions exist; for instance, in long spans at a river crossing, the cost of dead-end towers and foundations is a large item. Here, by using a smaller cable and sacrificing conductivity for a short distance, the stress is materially lessened and the cost of the supporting structures largely reduced.

The economies possible are evident, as in the case cited, where the cost of conductor was but 3 per cent of the total cost. Assume that by doubling its cost a lighter and stronger conductor can be obtained with no sacrifice in conductivity. This will reduce the necessary height of tower and weight of foundations, due to decreased sag, saving perhaps 10 per cent in cost of supports or a net saving of $(9.7 - 3) = 6.7$ per cent. With a lesser conductivity a greater saving might be made.

Occasionally the structural requirement is the most important consideration, as on special long-span construction just noted, or in some

districts liable to sleet and ice loading requiring a conductor large enough to withstand these loadings, or on light lines where a conductor large enough to withstand stresses is more than sufficient for transmission; in such cases the proper structural qualities are of most importance.

COST OF TRANSMISSION LINES

Wooden-pole Lines.—Following are data of cost, etc., of two wooden-pole lines:

1. Single circuit, No. 2 copper, 250-ft. spans; poles 35 ft. long with 8-in. top of western red cedar, butt treated; two cross arms, with ground wire at top cross arm; pin-type insulators.

Cost per mile, \$2500, including \$375 for average cost of right of way.

2. Single circuit, No. 3/0 A.C.S.R. wires, 450-ft. spans, capacity about 38,000 kw., insulated for 66,000 volts, spacing for 110,000 volts; H-frame, wood-arm, two-pole support with no provision for ground wire; semistrain type every $\frac{1}{2}$ mile, fully guyed every mile.

Cost per mile, \$6000, including \$560 for right of way.

Steel-tower Lines.—The cost of steel-tower transmission lines will evidently vary greatly, depending upon the conditions for which they are designed, particularly of sleet and wind loading.

C. R. Oliver¹ gives the following approximate data of cost:

A 110,000-volt line of flat construction, so called, with six wires in one plane, will cost about \$25,000 per mile, including cost of right of way, etc.

A 220,000-volt line, double circuit, cost about \$40,000 per mile. He further states that a line requisite to meet conditions in the New England Power Company district may cost \$15,000 per mile, as compared with perhaps \$8000 per mile in the South, under much less severe loading conditions.

For a 220,000-volt line of 150,000-kw. capacity, with one switching station every 100 miles, the cost per circuit as given in the report of the Northeastern Superpower Committee² is \$17,000, or \$34,000 per mile for a double-circuit line.

For a double-circuit standard tower line of from 20,000- to 30,000-kw. capacity at 66,000 volts, the approximate present cost is as follows per mile:

Line construction.....	\$12,000 to \$14,000
Right of way.....	3,000 to 4,000
Total cost.....	\$15,000 to \$18,000

Details of cost of a line of the Turners Falls Power and Electric Company follow. While this line was unusually costly, owing to construction conditions, the itemized statement gives a good idea of the relative costs of the different features.

¹ *Jour. A.I.E.E.*, November, 1925, p. 1177.

² "Superpower Studies of Northeast Section of the United States," p. 10, 1924.

COST OF TURNERS FALLS—PITTSFIELD TRANSMISSION LINE OF TURNERS FALLS
POWER AND ELECTRIC COMPANY
Constructed, June, 1922 to April, 1923

Description.—Two 66,000-volt circuits of 2/0 copper, on four-posted, galvanized-steel towers, with insulation and clearances suitable for future change to 110,000 volts. Steel ground wire, $\frac{3}{8}$ in. galvanized, fastened to peak of towers.

Length of line.....	36.7 miles
Towers per mile, average.....	9
Maximum span, Connecticut River.....	1,400 ft.
Maximum angle.....	45 deg. 28 min.
Right of way.....	100 ft. wide

Cost per Mile.

1. Right of way:		
a. Real estate and easements.....	\$3,097	
b. Legal and cost of purchase.....	523	
c. Clearing.....	1,339	
d. Finish and clean up.....	121	\$5,080
2. Towers:		
a. Footings.....	\$2,557	
b. Towers.....	2,577	5,134
3. Wire.....		2,298
4. Insulators.....		911
5. Erection:		
a. Towers.....	\$ 747	
b. Wire.....	944	
c. Freight and cartage.....	740	\$ 2,431
6. Miscellaneous.....		1,369
7. Telephone and carrier-current system.....		147
8. Special construction.....		1,056
9. Interest, taxes, and insurance during construction.....		1,333
10. Engineering and supervision.....		1,498
11. Total cost per mile.....		\$21,257

NOTES: Item 2a.—Cost of footings is high, as about half the tower stubs were set in rock, with much side-hill ledge construction. The cost of 17 swamp footings, with piles and concrete bases, is also included.

Item 2b.—Towers.

Kind	Number	Unit cost	Unit weight, pounds
Standard.....	271	\$219	3,878
Four circuit, joint with other companies..	45	384	6,766
Four circuit, for additional clearance.....	5	384	6,766
Special railroad.....	10		
Special river crossing.....	4		
Total.....	335		
Total weight of steel = 1,532,504 lb.			

Item 11.—The cost of this line was high, due largely to severe weather conditions, much of the work being done in winter; also some unusually difficult foundation conditions in portions of the line. The location was also unusually inaccessible.

Annual Costs.—The yearly allowance to cover fixed charges, maintenance, patrol, etc., of transmission lines, should be taken about as follows:

Wooden-pole lines.....	15 per cent
Steel-tower lines.....	12 per cent

CHAPTER XII

COST AND VALUE OF WATER POWER¹

Comparative Value of Water Power.—The comparative value or utility of any kind of power depends fundamentally upon cost of production. Owing to the very general use of steam power, which is ordinarily the kind of power with which water power must compete, the value of water power in a given locality is often determined by comparison with the cost of steam power, the comparison being made on the basis of the total annual cost per kilowatt-hour of each.

The elements entering into the cost of steam power are:

1. Fixed charges on plant and transmission costs, including interest, depreciation, taxes, etc.
2. Operating costs, including fuel, labor, maintenance and repairs, and miscellaneous items.

The elements of cost of water power are practically the same as for steam power, except that no fuel is required as an operating expense.

On the other hand, in amount these elements differ materially. Thus the first cost of a water power plant is often twice as large (or more) as that of the steam plant, resulting in greater fixed charges for the former. Labor and maintenance costs are much less for the water power plant, and the item of fuel also adds largely to the cost of steam power, depending upon the plant location with reference to fuel supply.

To offset this, the steam plant is usually located near the point of use of power, while water power must commonly be transmitted a considerable distance and, hence, at greater cost.

Broadly, the limit in allowable cost of a water power plant is reached when its fixed charges become so large as to offset its lower operating costs as compared to steam power, and for any locality, therefore, there is a limit in economical expenditure for a water power development. Conversely, a water power site which can be developed at low cost has a correspondingly high value as a water power privilege, reflecting the advantage in cost of power over that of steam.

In a large interconnected load system, however, steam and water power are usually desirable as complementary sources of power, and there is an economic balance between them or a "hydro ratio"

¹ As a supplement to this chapter for the further study of hydroelectric and steam-power economics the student should refer to Justin and Mervine, "Power Supply Economics," 1934.

TABLE 102.—WATER POWER PLANTS—COST ANALYSIS

Number	Date	Plant	State	Head, feet	Drainage area, square miles	Total horse power of plant capacity	Number of units	Cost per horse power, of wheel capacity, dollars	Per cent of total cost			Approximate cost index, per cent			Cost per horse power, dollars (cost index = 200 per cent)			Remarks
									Dam	Waterway or tail race	Power house and equipment	Dam	Waterway	Power house and equipment	Dam	Waterway	Power house and equipment	
1	1922	Lockwood Company	Me.	21	4,270	7,200	6	95	10	..	90	175	12	..	97	109	Concentrated fall	
2	1925	Vischer Ferry	N. Y.	26	3,430	8,000	2	191	42	..	58	200	80	..	111	191	Concentrated fall	
3	1925	Crescent	N. Y.	26	3,490	8,000	2	239	57	..	43	200	136	..	103	239	Concentrated fall	
4	1931	McIndoes Falls	N. H.-Vt.	27	2,200	16,500	4	105	34	2	64	205	34	3	64	101	Short tailrace	
5	1918	Ludlow	Mass.	40	686	5,750	3	125	36	..	64	185	48	..	88	136	Concentrated fall (short concrete flumes in dam)	
6	1923	Salmon Falls	N. H.	45	240	3,000	2	88	52	..	48	200	46	..	42	88	Concentrated fall	
7	1923	Amoskeag Company	N. H.	46	2,840	22,500	3	80	11	11	78	200	9	9	62	80	Tailrace about 1000 ft. long	
8	1924	Utilities Power Company	N. H.	53	740	7,500	3	95	62	..	38	215	55	..	34	89	Short steel penstocks in dam	
9	1932	Safe Harbor	Pa.	55	26,108	255,000	6	92	29	..	71	180	30	..	73	103	Concentrated fall	
10	1914	Turners Falls	Mass.	56	7,100	63,500	6	76	10	57	33	93	16	93	54	163	Canal about 2½ miles long, including 50-acre forebay	
11	1925	Rainbow	Conn.	56	600	12,000	2	96	48	11	41	200	46	10	40	96	Tailrace 1400 ft. long, automatic use	
12	1923	Lake Sunapee	N. H.	68	51	750	1	176	8	57	35	200	14	99	63	176	5 ft. wood-stave penstock 6000 ft.	
13	1916	Hiram	Me.	75	800	3,750	1	69	20	6	74	150	19	5	68	92	11 ft. wood-stave penstock 460 ft.	
14	1926	Sherman	Mass.	78	204	10,000	1	213	73	4	23	200	155	9	49	213	Steel penstock 200 ft., automatic use	
15	1926	Estes-Rainbow	N. Y.	89	500	5,600	3	67	25	21	54	200	16	21	35	65	Short canal and penstocks	
16	1930	Lower Fifteen Falls	N. H.-Vt.	170	1,600	215,000	4	77	60	8	32	207	45	6	24	75	Tailrace 6600 ft. long, 170 ft. dam, storage	
17	1921	Searsburg	Vt.	230	98	6,000	1	161	14	60	26	200	22	96	43	161	Penstock, wood-stave, 3½ miles, steel 400 ft.	
18	1923	Marlboro	N. H.	270	22	2,500	2	101	30	34	36	215	28	32	34	94	Penstock, wood-stave, 1.1 miles	
19	1914	Deerfield 5	Vt.	240	240	21,000	3	64	5	63	32	93	7	86	43	136	Tunnel and conduit, 1½ miles	
20	1926	Mollys Falls	Vt.	350	23	7,000	1	86	33	36	31	207	28	30	26	84	6 ft. wood-stave penstock 1½ miles, steel 1400 ft.	
21	1924	Davis Bridge (Harriman)	Vt.	360	184	60,000	3	162	52	32	16	215	78	48	24	150	Tunnel 2½ miles, steel penstock 600 ft., large storage reservoir included	
22	1913	Tallulah Falls	Ga.	580	190	85,000	5	47	33	33	34	100	31	31	32	94	Tunnel 1½ miles, steel penstock 1200 ft., storage reservoir included	
23	1924	Mystic Lake	Mont.	1,050	47	15,000	2	112	11	58	31	215	11	60	33	104	Tunnel 1000 ft., wood-stave penstock 9000 ft., steel pinstock 2730 ft.	

Note:

All installations are vertical units except: (18) Marlboro (one runner, horizontal), (19) Deerfield 5 (one runner, horizontal), (23) Mystic Lake (single-overhung horizontal Pelton). Transformers and high-tension equipment outside station except plants 1, 5, 6, 10, 12, 13, 19, and 23.

which will give the best results. In general, steam power can be used for base-load operation and water power for peak operation, as increment costs of water power are usually less than for steam power. Moreover, as has been shown (page 178), the upper portion of the load curve contains relatively little energy, and with pondage this can readily be carried by the hydroelectric plant, thus saving steam capacity.

Cost of Water Power Development.—The factors affecting the cost of a water power development have already been discussed on page 256 and may be summarized as:

1. The natural factors or conditions affecting construction and operating costs, or what have been called the "site characteristics."
2. The use and market characteristics as affecting the sale price and value of the power when developed.

Of the elements or essential features of a water power plant, the cost of the dam and waterway (and tailrace, if any) is affected most by the natural advantages or disadvantages of the site and usually constitutes the major portion of the plant cost except perhaps for low-head plants. The cost of the power house and its equipment is essentially dependent upon the character of load, particularly the load factor, and the use made of the power, constituting a more definite and usually smaller part of the plant cost than the features affected by natural conditions.

Analysis of Cost of Plants.—In Table 102 is given an analysis of cost of 23 plants varying in size from about 1000 to 255,000 hp. and for heads varying from 21 to 1050 ft. The cost of plant is given per horse power of wheel capacity, and also the proportionate cost of (1) dam, (2) waterway (or tailrace, if any), (3) power house and equipment. Costs of land and water rights are not included. For purposes of comparison, the plant cost per horse power is also adjusted to a cost-index number of 200 per cent. The cost-index numbers used are those of the *Engineering News-Record*, based on 1913 or prewar costs at 100 per cent (see page 682).

Based on these adjusted costs the total plant cost varies from \$65 to \$239 per horse power. A study of these costs indicates, as might be expected, a general tendency toward a lower cost per horse power as head and size of plant increase. The site characteristics are a more dominating feature, however, and wherever the cost of dam or waterway, or both, is high, the cost per horse power of the plant will be high, irrespective of cost of power house and equipment.

Considering the main elements of the plant, the variation in cost of plants listed in Table 102 is as shown in Table 103.

The average total cost of plant for the 23 listed is \$123 per horse power of capacity with a range (using group averages) from \$185 to \$81 per horse power. The group average costs for (a) 13 concentrated-fall plants of \$122 per horse power and (b) 10 divided-fall plants of \$124 per

TABLE 103.—SUMMARY OF PLANT COSTS FROM TABLE 102

Number in group	Plants included	Range of head, feet	Mean head, feet	Range of capacity, horse- power	Item	Cost per horse power of wheel capacity			Mean cost as a per cent of total
						Maxi- mum	Mini- mum	Mean	
A. Concentrated Fall (or short waterway or tailrace)									
4	1-4	21-27	25	7,200 — 16,500	Dam	136	12	65	43
					Power house and equipment	111	64	94	57
					Total	239	101	159	100
6	5-11 except 10	40-56	49	3,000 —255,000	Dam, etc.	55	18	40	42
					Power house and equipment	73	34	56	58
					Total	136	80	96	100
3	13, 14, 16	75-170	108	5,600 —215,000	Dam, etc.	155	19	73	58
					Power house and equipment	58	30	53	42
					Total	213	75	126	100
B. Divided Fall									
3	10, 12, 15	56-89	71	750 — 63,500	Dam	16	14	15	10
					Waterway	99	21	71	55
					Power house and equipment	63	35	50	35
					Total	176	65	136	100
6	17-22	230-580	340	2,500 — 85,000	Dam	78	7	33	27
					Waterway	96	30	55	45
					Power house and equipment	43	26	34	28
					Total	161	84	122	100
1	23		1050	15,000	Dam			11	11
					Waterway			60	58
					Power house and equipment			33	31
					Total			104	100
Mean 23 plants.....						185	81	123	
Mean 13 plants, concentrated fall.....						196	85	122	
Mean 10 plants, divided fall.....						168	75	124	

horse power are also practically the same in amount, with a slightly larger range (\$196 to \$85) for concentrated-fall plants than for divided fall plants (\$168 to \$75).

For *concentrated-fall* plants the costs of (1) dam and (2) power house and equipment are each about 50 per cent of the total cost, although for the lower heads this becomes nearer 60 per cent for power house and equipment and for the higher heads drops to about 40 per cent.

For *divided-fall* plants costs average approximately:

	Per Cent
Dam.....	20
Waterway.....	50
Power house and equipment.....	30
	100

Thus, broadly, the cost of dam for concentrated-fall plants and of dam plus waterway for divided-fall plants will normally include from 40 per cent of the total cost at low heads to 70 per cent of it at high heads. The remaining 60 per cent for low-head plants and 30 per cent for high-head plants will be cost of power house and equipment.

It must be kept in mind, however, that these are tendencies, based upon averages. Individual plant costs vary widely, as may be noted from Table 102.

Analysis of Power-house and Equipment Dimensions and Cost.

A further analysis of power-house and equipment dimensions and costs of the foregoing plants and for others is made in Table 104, where costs of substructure, superstructure, and total building are given (1) per square foot of area, (2) per cubic foot of volume, and (3) per horse power of wheel capacity. The area and volume of power house per horse power are also given, as well as the cost of power house and of equipment per horse power. Note that the volume used is that of superstructure alone, and that volume of substructure is not included. These costs are given revised in accordance with a 200 per cent cost-index number, as previously explained. In Table 105 is a summary of results from Table 104.

The impulse-wheel plant (No. 25) requires somewhat more space and volume per horse power of capacity than the high-head reaction-wheel plant, but this difference is relatively small.

Plotting relations from Table 105 the results in Table 106 were obtained.

The cost of power house and equipment thus varies between \$60 per horse power or more at low heads to about \$25 or less per horse power at high heads. The area per horse power will vary between 0.7 sq. ft. or more at low heads to 0.12 at high heads; the volume from about 30 cu. ft. to 5 cu. ft. per horse power. Here again it must be kept in mind that

TABLE 104.—POWER HOUSE—COST ANALYSIS

Number	Plant	Head, feet	Number of units	Horse power of wheels		Power house				Unit costs, dollars (Based on 200 per cent cost-index factor)										Character of building
				Unit	Total	Area, square feet	Volume, cubic feet	Area, square feet per horse power	Per square foot			Per cubic foot			Equipment per horse power	Power house per horse power	Power house and equipment per horse power			
									Substructure	Building	Superstructure	Substructure	Building	Superstructure				Total building		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
1	Holyoke 2	20	2	1,850	3,700	4,200	105,000	1.13	28.4	20.00	6.00	26.00	0.82	0.24	1.16	24.00	30.00	54.00	Brick	
2	Lockwood Company, Me.	21	6	1,200	7,200	5,500	190,000	0.76	26.4	30.20	18.80	49.00	0.87	0.55	1.42	39.80	37.40	77.20	Concrete	
3	Vischer Ferry, N. Y.	26	2	4,000	8,000	6,860	310,000	0.86	38.8	26.40	17.60	44.00	0.58	0.39	0.37	32.20	37.80	70.00	Brick	
4	Crescent, N. Y.	26	2	4,000	8,000	6,860	310,000	0.86	38.8	26.40	17.60	44.00	0.58	0.39	0.37	32.20	37.80	70.00	Brick	
5	McIndoes Falls	27	4	4,100	16,500	6,300	189,000	0.38	11.5	47.50	20.00	67.50	1.58	0.67	2.25	39.00	25.00	64.00	Brick	
6	Holyoke 8	30	1	6,000	6,000	1,400	40,000	0.23	6.7	48.00	6.30	54.30	1.65	0.22	1.87	17.00	12.50	29.50	Brick	
7	Ludlow, Mass	40	3	{ Two 2,600 } { One 550 }	5,750	5,850	246,000	1.01	42.8			49.00			1.16	38.00	50.00	88.00	Concrete and brick	
8	Salmon Falls, N. H.	45	2	2,800	3,000	2,120	91,300	0.70	30.4	12.00	12.00	24.20	0.28	0.28	0.56	24.60	17.10	41.70	Brick	
9	Amoskeag, N. H.	46	3	7,800	22,500	9,600	630,000	0.43	28.0	42.30	19.70	62.00	0.64	0.30	0.94	28.30	26.40	54.70	Brick	
10	Safe Harbor	53	6	42,500	255,000	105,000	10,250,000	0.41	40.0	47.00	18.50	65.50	0.48	0.19	0.67	45.70	26.80	72.50	Upstream concrete, downstream brick—interior tile face	
11	Utilities Power Company, N. H.	53	3	2,500	7,500	4,200	120,000	0.56	16.0			25.60			0.90	18.20	14.30	32.50	Brick	
12	Turners Falls, Mass.	56	6	10,500	63,000	23,200	1,000,000	0.37	16.0									55.50	Brick	
13	Rainbow, Conn.	56	2	6,000	12,000	4,000	132,000	0.33	11.0			15.00		0.36	1.22	24.00	15.00	39.00	Brick	
14	Lake Sunapee, N. H.	68	1	750	750	900	22,500	1.20	30.0			25.60			1.02	32.00	31.00	63.00	Brick	
15	Hiram, Me.	75	1	3,750	4,000	1,760	80,000	0.44	20.0							34.70	29.00	63.70	Concrete	
16	Sherman	78	1	10,000	10,000	2,600	83,000	0.26	8.3			49.50			1.55	36.00	13.00	49.00	Brick	
17	Estes-Rainbow, N. Y.	89	3	1,875	5,650	2,400	80,000	0.42	14.2							21.00	14.00	35.00	Concrete	
18	Lower Fifteen Mile Falls	170	4	54,000	216,000	22,400	1,340,000	0.11	6.7	32.80	50.00	82.80	0.55	0.84	1.39	16.10	8.60	24.70	Brick	
19	Searsburg, Vt.	230	1	6,000	6,000	1,100	28,000	0.18	4.7									41.30	Brick	
20	Marlboro, N. H.	240	2	1,250	2,500	1,200	23,000	0.48	9.2	13.50	11.80	25.30	0.71	0.63	1.34	14.90	12.20	27.10	Concrete	
21	Deerfield 5, Vt.	240	3	7,000	21,000	8,000	350,000	0.38	16.6							23.90	17.40	41.30	Brick	
22	Molly's Falls	350	1	7,000	7,000	940	32,000	0.13	4.6			54.50			1.60	12.00	7.30	19.30	Brick	
23	Davis Bridge, Vt.	360	3	20,000	60,000	6,800	306,000	0.11	5.1	17.50	21.20	38.70	0.31	0.39	0.70	20.00	4.40	24.40	Brick	
24	Tallulah Falls, Ga.	580	5	17,000	85,000	19,000	1,500,000	0.22	17.5							23.00	10.80	33.80	Brick	
25	Mystic Lake, Mont.	1,050	2	7,500	15,000	5,260	284,000	0.35	19.0			37.00			0.69	22.00	13.00	35.00	Concrete	

NOTE: The volume used in cols. 8, 14, 15, and 16 is that of superstructure only.

TABLE 105.—SUMMARY OF POWER-HOUSE AND EQUIPMENT DATA—FROM TABLE 104

Item	1			2			3			4			5			6
	Group			1-6			7-13			14-17			18-21			22-24
	Number			6			7			4			4			3
	Maxi- mum	Mini- mum	Aver- age	Maxi- mum	Mini- mum	Aver- age	Maxi- mum	Mini- mum	Aver- age	Maxi- mum	Mini- mum	Aver- age	Maxi- mum	Mini- mum	Aver- age	
Capacity, horse power.....	16,500	3700	8200	255,000	3000	53,000	10,000	750	5100	216,000	2500	61,000	85,000	7000	50,000	15,000
Head, feet.....	30	20	25	56	40	50	89	68	78	240	170	220	580	350	430	1050
<i>Per horse power of capacity:</i>																
Power-house superstructure:																
Area, square feet.....	1.13	0.23	0.70	1.01	0.33	0.54	1.20	0.26	0.58	0.48	0.11	0.30	0.22	0.11	0.15	0.35
Volume, cubic feet.....	38.8	6.7	23.4	42.8	11.0	26.3	30.0	8.3	18.1	16.6	4.7	9.3	17.5	4.6	9.0	19.0
<i>Cost, dollars per horse power:</i>																
Building.....	41	13	30	50	14	25	31	13	22	17	9	14	11	4	7	13
Equipment.....	40	17	30	46	18	30	36	21	31	24	15	20	23	11	18	22
Total.....	77	30	60	88	32	56	64	35	52	41	24	34	34	14	25	35

these are average figures and individual plants will show considerable variations from them.

TABLE 106.—POWER-HOUSE COSTS AND DIMENSIONS

Head, feet	Cost per horse power of capacity			Area per horse power, square feet	Volume per horse power, cubic feet
	Building	Equipment	Total		
25	\$29	\$31	\$60	0.63	26
50	25	30	55	0.57	21
100	21	26	47	0.47	16
200	15	21	36	0.33	12
300	11	18	29	0.23	10
400	8	17	25	0.17	8
500	6	16	22	0.12	7

Of a cost of building of \$25 per horse power, about \$15 will represent substructure and \$10 superstructure. The equipment cost of \$25 per horse power may be segregated into:

Hydraulic and miscellaneous..... \$10

Electrical:

Generators..... \$7

Transformers..... 2

Switching equipment and wiring..... 6 15

Total equipment cost..... \$25 per horse power

Hydraulic and miscellaneous equipment may range in cost from \$3 to \$15 per horse power and electrical equipment from \$10 to \$20 per horse power (of wheel capacity).

Cost of Water Power.—As previously noted, the items of cost for water power are (1) fixed charges and (2) operating costs.

1. *The fixed charges* will include:

	Per Cent
Interest.....	6- 7
Depreciation.....	2- 4
Taxes and insurance.....	1- 2
Total.....	9-13

Usually from 9 to 12 per cent may be assumed as total fixed charges, depending upon the circumstances under which the plant must be built and the type of construction. A rate often used is 10 per cent, based on a 6 per cent rate of interest or value of money which represents the average actual annual cost of the capital involved. Public utilities are usually allowed an 8 per cent return on capital, or 2 per cent over the cost of the money, in order to allow a profit that will be attractive to capital and

TABLE 107.—ASSUMED LIFE AND DEPRECIATION ALLOWANCES

Item	Assumed life, years	Depreciation allowance, per cent yearly (straight-line basis)
Dam:		
Concrete:		
Solid.....	50-100	2-1
Reinforced.....	25-50	4-2
Earth.....	50-100	2-1
Waterway:		
Canals:		
Unlined.....	50-100	2-1
Lined.....	50-100	2-1
Penstocks:		
Tunnel.....	50-100	2-1
Wood stave.....	20-30	5-3 $\frac{1}{3}$
Concrete.....	25-50	4-2
Steel.....	40-50	2 $\frac{1}{2}$ -2
Power house and equipment:		
Building:		
Concrete.....	50	2
Brick.....	50	2
Equipment:		
Wheels.....	20-30	5-3 $\frac{1}{3}$
Generators.....	20	5
Miscellaneous.....	30	3 $\frac{1}{3}$

DEPRECIATION ALLOWANCES—TURNER FALLS AND SEARSBURG PLANTS

Item	Per cent of total cost	Allowance, per cent	Weighted depreciation, per cent
Turners Falls:			
Dam, concrete.....	10	2	0.20
Waterway, lined canal.....	57	2	1.14
Power house:			
Building and equipment.....	33	2 $\frac{1}{4}$ } 3	1.00
Total.....	100		2.34
			say, 2.3 per cent yearly
Searsburg:			
Dam, earth and concrete.....	14	2	0.28
Waterway, wood-stave pipe (mostly).....	60	5	3.00
Power house and equipment..	26	3	0.78
Total.....	100		4.06
			Say, 4 per cent yearly

thus stimulate an interest in the industry. In this case 12 per cent is a fair average allowance for fixed charges, allowing a somewhat higher rate of return on the investment.

In considering the item of depreciation, it is desirable to make a careful study to determine the proper allowance, based upon suitable periods of use (or life) of the various elements of which the plant consists. In Table 107 shown on page 648, are listed some of the principal items making up a typical plant, with common values assumed for life and the corresponding depreciation allowances.

Thus, for the Turners Falls and Searsburg plants in Table 102, the proper allowance for depreciation would be computed as shown in table at foot of page 648 (using the shorter periods of life in Table 107):

For these two plants, assuming interest at 6 per cent and taxes, etc., at 2 per cent, the yearly allowance for fixed charges would be

	Per Cent
Turners Falls.....	10.3
Searsburg.....	12.0

the difference being due chiefly to the long wood-stave penstock at Searsburg, with its much greater depreciation requirements.

The determination of the yearly depreciation allowance is also often made by the "sinking-fund" method. This provides for setting aside each year such a sum that, invested at a certain interest rate compounded annually (or semiannually), it will equal the amount of the depreciable property at the end of its life. It requires smaller yearly amounts than the straight-line method, and the amounts are uniform.

Thus, if S = the amount of a sinking fund,

n = number of years of its creation,

i = interest rate expressed as a decimal,

A = annual installment paid at the end of each year,

then
$$A = \frac{Si}{(1+i)^n - 1}$$

If S is taken as \$1 then A will be the yearly depreciation rate (expressed as a decimal).

A comparison of depreciation allowances by the straight-line and sinking-fund methods is shown at top of page 650.

For the typical hydroelectric plant the elements of which will usually have a life of 20 to 50 years, it will be seen that depreciation by the sinking-fund method will be approximately 1 to 1.5 per cent less than by the straight-line method.

The fixed charges per kilowatt-hour will depend upon the annual output as well as the fixed charges. Using a capacity factor of 0.40, the yearly output per horse power of plant capacity would be (with generator efficiency of 94 per cent)

COMPARISON OF DEPRECIATION ALLOWANCES

Assumed life of structure, years	Yearly depreciation allowance, per cent			
	Straight- line method	Sinking-fund method		
		$i = 5\%$	$i = 6\%$	$i = 4\%$
10	10.00	7.95	7.60	8.33
15	6.67	4.63	4.30	4.99
20	5.00	3.02	2.72	3.36
30	3.33	1.51	1.27	1.78
40	2.50	0.83	0.65	1.05
50	2.00	0.48	0.34	0.65

$$1 \times 8760 \times 0.746 \times 0.94 \times 0.40 = 2450 \text{ kw.-hr.}$$

On this basis, fixed charges would be as follows for plants of different cost:

TABLE 108.—FIXED CHARGES PER KILOWATT-HOUR FOR DIFFERENT ASSUMED PLANT COSTS

(Yearly output for 1 hp. of capacity assumed at 2450 kw.-hr.)

Assumed cost of plant per horse power (1)	Fixed charges, 10 per cent of Col. 1 (2)	Fixed charges, cents per kilowatt-hour (3)
50	\$ 5.00	0.20
100	10.00	0.41
150	15.00	0.61
200	20.00	0.82
250	25.00	1.02
300	30.00	1.22
350	35.00	1.43
400	40.00	1.63

2. *Operating costs* will consist chiefly of wages, with a small allowance for minor repairs and miscellaneous expenses.

A hydroelectric station of ordinary size, such as one between 5000 and 10,000 kw. with a yearly output of 20 to 40 mill. kw.-hr. will require three shifts of two to three men each and also a chief operator, whose wages will total about \$15,000 per year. The cost for labor would thus vary from \$1.50 to \$3 per year per kilowatt of capacity, depending upon size of plant and character of service, or from 0.0375 to 0.075 ct. per kilowatt-hour. A large plant will require more attendants but with a smaller proportionate cost per kilowatt-hour owing to its increased capacity. A reasonable range for cost of attendance is from 0.025 to 0.1 ct. per kilowatt-hour, including allowance for small repairs and

miscellaneous expenses as well, and, unless the plant is of small capacity, this item of operating costs will not usually exceed 0.05 ct. per kilowatt-hour.

It is obvious, therefore, that the fixed charges constitute most of the cost of water power, and this becomes more nearly true for large plants.

Total Cost of Water Power.—Allowing 0.05 ct. per kilowatt-hour for operating costs and adding this amount to fixed charges as given in Table 108 gives the total cost of water power for a plant having a capacity factor of 0.4. These figures of cost are shown in Table 109, where are given similar costs for other capacity factors through their ordinary range of value.

TABLE 109.—COST OF WATER POWER PER KILOWATT-HOUR FOR DIFFERENT ASSUMED PLANT COSTS AND CAPACITY FACTORS

Assumed cost of plant per horse power	Capacity factor					
	0.20	0.30	0.40	0.50	0.60	0.70
	Cost in cents per kilowatt-hour (fixed charge and operating costs)					
\$ 50	0.45	0.32	0.25	0.21	0.19	0.17
100	0.86	0.59	0.46	0.37	0.33	0.28
150	1.26	0.86	0.66	0.53	0.47	0.39
200	1.67	1.13	0.87	0.70	0.61	0.51
250	2.07	1.40	1.07	0.85	0.75	0.62
300	2.48	1.67	1.27	1.01	0.88	0.74
350	2.91	1.93	1.48	1.18	1.02	0.85
400	3.29	2.20	1.68	1.34	1.16	0.97

The costs in Table 109 are those at the switchboard and include no allowance for transmission costs.

These costs show particularly the great importance of capacity factor upon the cost of water power. For example, a plant costing \$220 per horse power, if operated on a 40 per cent capacity factor, will produce power at about 1.0 ct. per kilowatt-hour or a relatively high cost, whereas with a high capacity factor, as 70 per cent, this cost will be less than 0.6 ct. per kilowatt-hour, which is comparatively low.

Effect of Capacity Factor on Cost of Water Power Plant.—While a low capacity factor means a larger and more expensive water power plant, this increase in cost is not in inverse proportion to capacity factor. Normally the cost of dam and waterway will constitute a large portion of the plant cost, and the cost of these features in low-head plants is not usually affected by capacity factor. Occasionally, with a very low capacity factor, the location or cross section of the waterway, if a canal, might be controlled by maximum velocity conditions and, hence, its cost perhaps

slightly affected by capacity factor. Usually, however, the capacity of waterway is fixed by the average rather than the maximum loss in head, and its cost is, therefore, substantially independent of capacity factor, at least with the usual form of load curve.

The remaining cost—of power house and equipment—will vary approximately inversely with the capacity factor. This is not quite true, because the increased capacity required with a low capacity factor will not usually mean a larger number of units but rather an increased size of unit, for which the cost per horse power may be a little less. Similarly, the power house will not have to be twice as great for a 40 per cent capacity factor as for an 80 per cent capacity factor.

Assuming, however, for the purpose of comparison, that power-house and equipment costs vary inversely as capacity factor, for ordinary values of these costs and plant costs, the effect of various capacity factors on cost of plant to deliver 1 hp. is shown in the following table:

TABLE 110.—EFFECT OF CAPACITY FACTOR ON COST OF WATER POWER PLANTS

Cost of plant to deliver 1 hp. on wheel shaft with different capacity factors						Per cent increase in cost over that for 100 per cent capacity factor		
100 per cent capacity factor			Total cost					
Power house and equip- ment	Dam, water- way, etc.	Total cost	80 per cent capacity factor	50 per cent capacity factor	30 per cent capacity factor	80 per cent capacity factor	50 per cent capacity factor	30 per cent capacity factor
\$ 25	\$ 45	\$ 70	\$ 76	\$ 95	\$128	8	36	83
30	70	100	107	130	170	7	30	70
70	130	200	217	270	364	8	35	82
100	100	200	225	300	433	12	50	116
70	230	300	317	370	464	6	23	55
100	200	300	325	400	533	8	33	77

For the range of costs considered, the increase in cost and capacity of plant over that with 100 per cent capacity factor is:

Capacity factor, per cent	Increase in cost, per cent	Increase in plant capacity, per cent
80	6- 12	25
50	23- 50	100
30	55-116	233

For high capacity factors this increase in cost is small, but with very low capacity factors the cost of the plant may be doubled, keeping in

mind, however, that the additional capacity is often obtained at less than 50 per cent of the original unit cost. Thus the same general conclusion as to the effect of capacity factor on cost will be reached whether a given plant operating at different capacity factors is considered or a given average load is assumed and the size of plant assumed to vary with capacity factor.

Transmission Costs.—In determining the relative economy of water power as well as steam power in some cases, where large units are at some distance from the load center, it is often necessary to allow for the cost of transmission of power to the point of use. Thus, in the Atlantic states hydroelectric power from the Conowingo plant used in Philadelphia or from the Safe Harbor plant used in Baltimore must carry the transmission costs from plants to load centers. On the other hand, the Holland steam plant on the Delaware River is also some distance from its load centers and is similarly subject to some transmission charges.

The cost and other features of power transmission are given in detail in Chap. XI.

A 100-mile high-voltage double-circuit transmission line of, say, 150,000-kw. capacity under usual conditions will cost about \$40,000 per mile or \$4,000,000 total cost. The cost per kilowatt of capacity would be about \$27. Yearly costs would be about 15 per cent of \$27 or \$4 per kilowatt-year of capacity, to which would have to be added the cost of operating and patrolling the line. For large plants, requiring from 50 to 150 miles of additional transmission for their exclusive use, transmission cost may vary from about \$2.50 to \$7 per kilowatt-year of installed capacity or for a 60 per cent capacity factor from about 0.05 to 0.15 ct. per kilowatt-hour.

Cost of Steam Power. *Cost of Plant.*—In recent years there has been a great advance in the art of generating electric power from steam. During the past decade the average quantity of coal used per kilowatt-hour has been reduced about one-half (see Table 8, Chap. I) and is now about 1.5 lb. New plants which can generate 1 kw-hr. with 1 lb. of coal are common and some high-pressure plants do considerably better. While some further improvement is likely, it is improbable that fuel cost for steam power will continue to decline as rapidly as it has in the immediate past.

The cost of a modern steam plant may vary between a range of \$80 to \$180 per kilowatt of capacity. The highest unit costs are usually caused by provision for future increase of capacity and will lower materially as these increases are made.

A fair range of cost of steam plant of \$100 to \$125 per kilowatt may usually be assumed in comparisons of steam and hydroelectric costs.

Detailed costs for a recent steam-electric plant of the Montaup Electric Company are given in Table 111. As will be noted, this plant

as constructed in 1923 had one 32,000-kw. unit and cost about \$192 per kilowatt; with a second unit installed later, the cost was reduced to about \$132 per kilowatt.

TABLE 111.—COST OF SOMERSET STEAM STATION, MONTAUP ELECTRIC COMPANY, FALL RIVER, MASS.¹

Item	As constructed, 1923, one unit = 32,000 kw.		With additional unit two units = 64,000 kw.	
	Cost, total	Cost per kilowatt	Estimated cost, total	Cost per kilowatt
Land, water supply, yard work, etc.	\$1,203,427	\$ 37.60	\$1,203,427	\$ 18.80
Building and foundations.....	1,562,136	48.80	1,562,136	24.40
Equipment.....	3,383,361	105.70	5,697,674	89.00
Total.....	\$6,148,924	\$192.10	\$8,463,237	\$132.20

¹ *Elec. World*, vol. 83, Mar. 8, 1924, pp. 475-476.

Fixed Charges for Steam Plants.—Fixed charges for a steam-electric plant should be taken at a somewhat higher rate than for the water power plant, owing to the greater proportion of equipment and machinery cost in the former, for which depreciation charges are relatively high. Ordinarily 12 to 14 per cent would be a fair figure for the steam-plant fixed charges, which would, therefore, vary as follows for plants of varying cost and capacity factor:

TABLE 112.—FIXED CHARGES IN CENTS PER KILOWATT-HOUR FOR STEAM-ELECTRIC PLANTS

Cost per kilowatt of plant	Fixed charges at 12 per cent				Fixed charges at 14 per cent			
	Capacity factor, per cent				Capacity factor, per cent			
	100	75	50	25	100	75	50	25
50	0.07	0.09	0.14	0.27	0.08	0.12	0.16	0.32
100	0.14	0.20	0.27	0.55	0.16	0.24	0.32	0.64
150	0.21	0.27	0.41	0.82	0.24	0.36	0.48	0.96
200	0.27	0.41	0.55	1.10	0.32	0.48	0.64	1.28

Fixed charges may, therefore, vary from about 0.10 ct. per kilowatt-hour for low-cost plants, with high capacity factors, up to 0.5 to 1.3 ct. per kilowatt-hour for plants whose cost is high and whose capacity factors are low.

Operating Costs.—The important operating costs are those of fuel and labor or attendance. Coal consumption varies from small-plant use

of 4 or 5 lb. per kilowatt-hour to about 1.5 lb. per kilowatt-hour for a very efficient reciprocating engine. Steam turbo-units of 1000-kw. capacity require from 2 to $2\frac{1}{2}$ lb., while larger units may range from 2 to 1 lb. per kilowatt-hour, or even less. Labor costs will vary from 0.1 to 0.3 ct. per kilowatt-hour, depending principally upon the plant capacity factor.

In comparing steam and hydroelectric costs, it is common to divide operating costs into

1. Fixed cost expressed usually in dollars per kilowatt-year—including items of attendance, etc., which are substantially constant in amount regardless of peak load or output.

2. Increment cost per kilowatt-hour—a practically constant unit cost involving chiefly cost of fuel.

On this basis, operating costs for modern steam plants of large capacity will range from

Fixed cost: \$5 to \$10 yearly per kilowatt of capacity.

Increment cost: 0.2 to 0.3 ct. per kilowatt-hour, which would correspond approximately to a cost of coal ranging from \$4 to \$6 per ton, assuming a use of 1 lb. of coal per kilowatt-hour.

Thus, with a 60 per cent capacity factor (5250 hr. yearly) the cost per kilowatt-hour would be:

Fixed cost.....	0.1-0.2 ct.
Increment cost.....	0.2-0.3
Total operating cost.....	0.3-0.5 ct.

Total Cost of Steam Power.—The total cost of steam power for a large modern plant, for comparison with hydroelectric costs, is given in Table 113 based upon the following assumptions:

Plant cost: (1) \$100, (2) \$125, and (3) \$150 per kilowatt of capacity.

Fixed charges: 12 per cent yearly.

Operating costs—fixed: \$6 yearly per kilowatt of capacity. Increment costs, 0.2 and 0.3 ct. per kilowatt-hour (which correspond approximately to \$4 coal and 1 lb. of coal per kilowatt-hour and to \$4 coal and 1.5 lb. of coal per kilowatt-hour or \$6 coal and 1 lb. per kilowatt-hour, respectively).

Capacity factors: 100 to 20 per cent.

Limits in Economy of Water Power.—A comparative study of variation in cost of water power, as shown by Table 109, and of steam power by Table 113 and Fig. 281 will indicate the futility of attempting to fix a definite general limit in economical expenditure for a water power plant in comparison with steam power because of the great variation in cost of the latter in different localities, due not only to variable fuel cost but also to the load conditions under which the plant must operate and

whether an industrial steam plant or large modern central station is considered. Furthermore, in such a comparison it is the average future price of coal—a problematical quantity—which is of importance, as well as future development in steam-plant design, which may tend to lower costs somewhat below what is now possible.

TABLE 113.—TOTAL COST OF STEAM-ELECTRIC POWER IN CENTS PER KILOWATT-HOUR

Capacity factor, per cent	Plant cost, \$100 per kilowatt		Plant cost, \$125 per kilowatt		Plant cost, \$150 per kilowatt	
	Increment operating cost per kilowatt-hour		Increment operating cost per kilowatt-hour		Increment operating cost per kilowatt-hour	
	0.2 ct.	0.3 ct.	0.2 ct.	0.3 ct.	0.2 ct.	0.3 ct.
100	0.41	0.51	0.44	0.54	0.48	0.58
70	0.50	0.60	0.55	0.65	0.59	0.69
60	0.54	0.64	0.60	0.70	0.66	0.76
50	0.61	0.71	0.68	0.78	0.75	0.85
40	0.72	0.82	0.80	0.90	0.89	0.99
30	0.88	0.98	1.00	1.10	1.13	1.23
20	1.22	1.32	1.40	1.50	1.58	1.68

Cost of Coal.—The average price of bituminous coal f.o.b. cars at the mines¹ between the years 1880 and 1914 was about \$1 per net ton, varying from \$0.80 in 1898 to \$1.24 in 1903. For the years 1915–1931 it was as follows:

Year	Cost	Year	Cost
1915	\$1.13	1924	\$2.20
1916	1.32	1925	2.04
1917	2.26	1926	2.06
1918	2.58	1927	1.99
1919	2.49	1928	1.80
1920	3.75	1929	1.79
1921	2.89	1930	1.70
1922	3.02	1931	1.54
1923	2.68		

In 1922 the cost of coal at the mines of approximately \$3 was about \$2 per ton in excess of prewar costs. The 1931 cost of \$1.54 was about \$0.50 in excess of prewar costs and one-half the cost of 1922.

Of the bituminous coal mined in the United States, 74 per cent comes from four states, West Virginia, Pennsylvania, Illinois, and Kentucky, as indicated by the following table showing production and cost for 1931:²

¹ "Coal in 1927 and 1931," U. S. Dept. Commerce.

² *Ibid.*

TABLE 114.—BITUMINOUS COAL PRODUCTION AND COST, 1931

State	Net million tons	Per cent of total in United States	Average cost per ton at mines
West Virginia.....	101.5	26.5	\$1.31
Pennsylvania.....	97.6	25.5	1.59
Illinois.....	44.3	11.6	1.70
Kentucky.....	40.0	10.5	1.34
Ohio.....	20.4	5.3	1.24
Indiana.....	14.3	3.7	1.45
Alabama.....	12.0	3.1	1.82
Virginia.....	9.7	2.5	1.45
Colorado.....	6.6	1.7	2.41
Total nine states.....	346.4	90.4	
Total, United States.....	382.1	100.0	1.54

Obviously, the cost of coal at market or at the steam plant is largely freight or transportation cost, and the cost of coal will vary chiefly with distance from the mines and transportation facilities.

Abbot¹ suggested that freight rates for coal in the Middle West could be approximated on the basis of 0.4 ct. per (gross) ton-mile, plus a base rate of \$1 per ton. For the East this formula gives too low results, and a rate of about 8 mills per (gross) ton-mile applies more nearly where shipped by rail from the Clearfield, Pennsylvania, district. By rail and tidewater, from the West Virginia (New River) district to New England ports the cost is about 10 per cent less in vessel at point of delivery.

Assuming a freight rate of 8 mills per ton-mile and that 25 cts. per ton will cover cost of unloading and placing ready for use at the boilers, coal costs would be as follows for a base cost of \$1.50 for various distances from the mines:

TABLE 115.—COST OF COAL PER NET TON AT POWER PLANT IN EASTERN UNITED STATES

Distance from mine, miles	Cost, f.o.b. at mines	Freight cost	Cost of delivery to power house	Total cost per ton
250	\$1.50	\$1.80	\$0.25	\$3.55
500	1.50	3.60	0.25	5.35
750	1.50	5.40	0.25	7.15

¹ ABBOT, W. L., "Power Development in the Middle West," *Trans. A.S.C.E.*, 1925, p. 237.

The cost of coal at the power plant in the East will thus vary between about \$3.50 and \$7 per net ton, depending upon distance from the mines. In the central states, using coal from the local mines, the cost is from about \$2.50 to \$3.50 per net ton, depending upon the locality.

Comparative Economy—Water and Steam Power.—The cost of water power has been given in Table 109, and of steam power in Table 113,

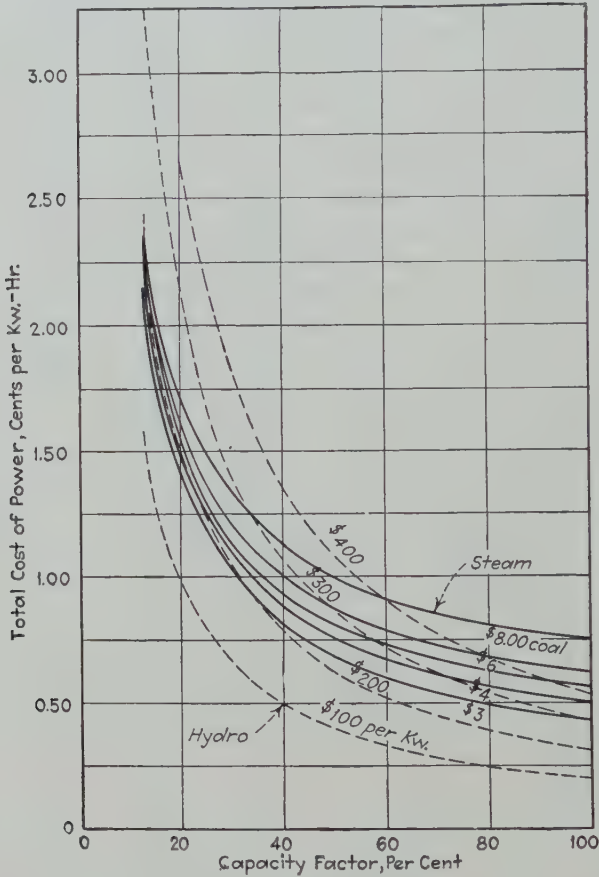


FIG. 281.—Cost of steam and hydroelectric power.

for different plant costs and capacity factors. As a means of comparison of these costs, Fig. 281 is convenient, where cost per kilowatt-hour of power is plotted against capacity factor; the full lines show cost of steam power for a plant cost of \$112 per kilowatt of capacity and for various costs of coal per ton of 2000 lb., and the dotted lines show cost of hydroelectric power for various plant costs per kilowatt of capacity.

In using this diagram it should be kept in mind that the character of the hydroelectric power should be taken into account and allowance

made for cost—as of storage or auxiliary power required, if any, to make this of firm capacity, comparable with the steam power in performance.

For example, consider a hydroelectric plant upon a river with flow-duration curve similar to the mean curve in Fig. 45 with plant cost of \$150 per kilowatt with wheel capacity of about 1.8 sec.-ft. per square mile, used in a system with a 60 per cent load factor, a drainage area of 715 sq. miles and a head of 100 ft.:

Wheel capacity	=	1,300 sec.-ft.
	=	12,000 hp. under 100-ft. head
	=	8,400 kw.
Average flow utilized with 85 per cent utilization factor	{	= 0.73 sec.-ft. per square mile
	=	520 sec.-ft.
	=	4,850 hp. under 100-ft. head
	=	3,400 kw.
	=	30 mill. kw.-hr. yearly
Output of 12,000-hp. steam plant with 60 per cent capacity factor	{	= 7,200 hp.
	=	5,000 kw.
	=	43.5 mill. kw.-hr. yearly
Power from storage required by hydroelectric plant	{	= 13.5 mill. kw.-hr. yearly

Assuming stored water can be supplied at a cost of 0.6 ct. per kilowatt-hour (or a little over \$1 per million cubic feet per foot of fall, used once during a season—see page 151), the added yearly cost for storage would be

$$13,500,000 \times 0.6 \text{ ct.} = \$81,000$$

or
$$\frac{\$81,000}{8400} = \$9.70 \text{ per kilowatt-year of capacity.}$$

Capitalized at 10 per cent this is about \$97 per kilowatt, which added to the plant cost of \$150 would make \$247 per kilowatt as the cost of the hydroelectric plant to equal the performance of the steam plant.

From Fig. 281 for a hydroelectric-plant cost of \$247 per kilowatt and 60 per cent capacity factor, the cost of power would be about 0.55 ct. per kilowatt-hour. Steam power on the same capacity factor with coal at \$4 per ton would cost about 0.7 ct. per kilowatt-hour, the advantage in this case lying with the hydroelectric plant.

From Fig. 281 it appears that for the ordinary range of capacity factor the approximate limits in cost of hydroelectric plant (including capitalized cost of auxiliary power or storage, if any) in order not to exceed the cost of steam power are about as follows for different fuel costs:—

Cost of Coal per Ton	Limiting Cost of Hydroelectric Plant per Kilowatt of Capacity
\$3.00	\$200 to \$250
4.00	250 to 300
5.00	250 to 350
6.00	275 to 400

Figure 281 should be used only as an approximate guide for preliminary determinations of relative economy of hydroelectric and steam plants. Each case should be studied by itself and a complete comparative analysis made, which takes into account all essential factors.

Use of Hydroelectric Power for Peak Loads.—With the lower load factors firm or primary *capacity* of water power can be increased by pondage so that even a run of stream plant may show a primary capacity several times the 24-hr. minimum power. (This general phase of hydroelectric use has already been discussed on page 177.) As an example consider the same 12,000-hp. hydroelectric and steam plants in a system with 60 per cent load factor as before, and assume that the primary 24-hr. average river flow is 0.25 sec.-ft. per square mile, or 178 sec.-ft., and that ample pondage is available at the hydroelectric plant.

The available primary 24-hr. power is about 1660 hp., which is $1660/7200 = 0.23$ of the required average daily energy output. Referring to Table 54, page 179, it will be seen that for a 60 per cent load factor, 0.23 of the daily energy lies in that part of the load curve between about 54 and 100 per cent of the peak.

Hence the hydroelectric plant with pondage can supply from the minimum flow of 178 sec.-ft., about 46 per cent of the peak load, or about 5500 hp. The remaining 54 per cent or 6500 hp. of capacity would, in the case of the hydroelectric plant, have to be supplied by auxiliary steam or other power to give the same service as the steam plant.

The hydroelectric plant would, during the average year, supply an average of about 4850 hp. with a primary capacity of $0.46 \times 12,000 = 5500$ hp. The yearly output would be about 30 mill. kw.-hr., and about 13.5 mill. kw.-hr. of auxiliary power would be required to equal the performance of the steam plant. As previously noted, auxiliary capacity required would be about 6500 hp., and the auxiliary power would have to be supplied upon a capacity factor of about 0.34.

Assuming further as before that the hydroelectric plant costs \$150 per kilowatt of capacity and has yearly fixed charges of 10 per cent and an operating charge of 0.04 ct. per kilowatt-hour, the cost of water power would be:

Fixed charges.....	$\$150 \times 8400 \times 0.10$	= \$126,000
Operating charges.....	34.5 mill. kw.-hr. at 0.04 ct.	= 14,000
Total yearly cost.....		\$140,000

Auxiliary steam power on a 34 per cent capacity factor, assuming a plant cost of \$100 per kilowatt, would cost about 0.9 ct. per kilowatt-hour (see Table 113) for 13.5 mill. kw.-hr., or \$121,000, making a total yearly cost of hydroelectric plant and auxiliary of \$261,000

or $\frac{\$261,000}{43.5} = 0.60$ ct. per kilowatt-hour, all primary.

A steam plant of 12,000-hp. capacity under 60 per cent capacity factor at an installation cost of \$125 per kilowatt would produce power at about 0.6 to 0.7 ct. per kw.-hr. (see Table 113) or a little more than the equivalent hydroelectric plant with auxiliary steam.

There is some advantage in this case, therefore, in the economy of the hydroelectric plant, with its auxiliary, and that of the steam plant.

The pondage required in the above case, at the time of low water, would be for a period of about $(24 - 13) = 11$ hr. (Col. 5, Table 54) or about $178 \times \frac{11}{2} = 162$ acre-ft.

The above problem, solved by the use of Table 54, illustrates the manner in which hydroelectric power may be used to carry peak loads. In most cases it will be desirable to use actual load curves for the entire week of maximum yearly peak instead of the single day and prepare a curve of per cent of daily energy plotted against per cent of peak to use in place of the approximate values in Cols. 3, 6, and 9 of Table 54. The general method of procedure will, however, be similar to that outlined in the above problem.

TABLE 116.—SYSTEM LOAD-CURVE CHARACTERISTICS OF THE NEW YORK EDISON AND UNITED COMPANIES FOR THE YEAR 1922

Period	Condi- tions	Day	Maximum load in per cent yearly peak	Average load in per cent of yearly peak	Daily load factor, per cent	Per cent of daily load above average
Winter, November-January	Cloudy Clear	Week	100	53	53	21
		Week	100	50	50	21
		Sunday	48	27	56	17
Weighted mean.			92	49	52	20
Spring, February-April. .	Cloudy Clear	Week	74	47	61	22
		Week	59	39	67	18
		Sunday	40	22	55	15
Weighted mean.			63	40	63	19
Summer, May-July.	Cloudy Clear	Week	69	40	58	18
		Week	55	37	68	17
		Sunday	36	21	58	9
Weighted mean.			58	36	62	16
Fall, August-October. . .	Cloudy Clear	Week	78	49	63	20
		Week	73	44	60	19
		Sunday	43	25	58	14
Weighted mean.			71	43	61	19
Year, mean.			70	42	60	19

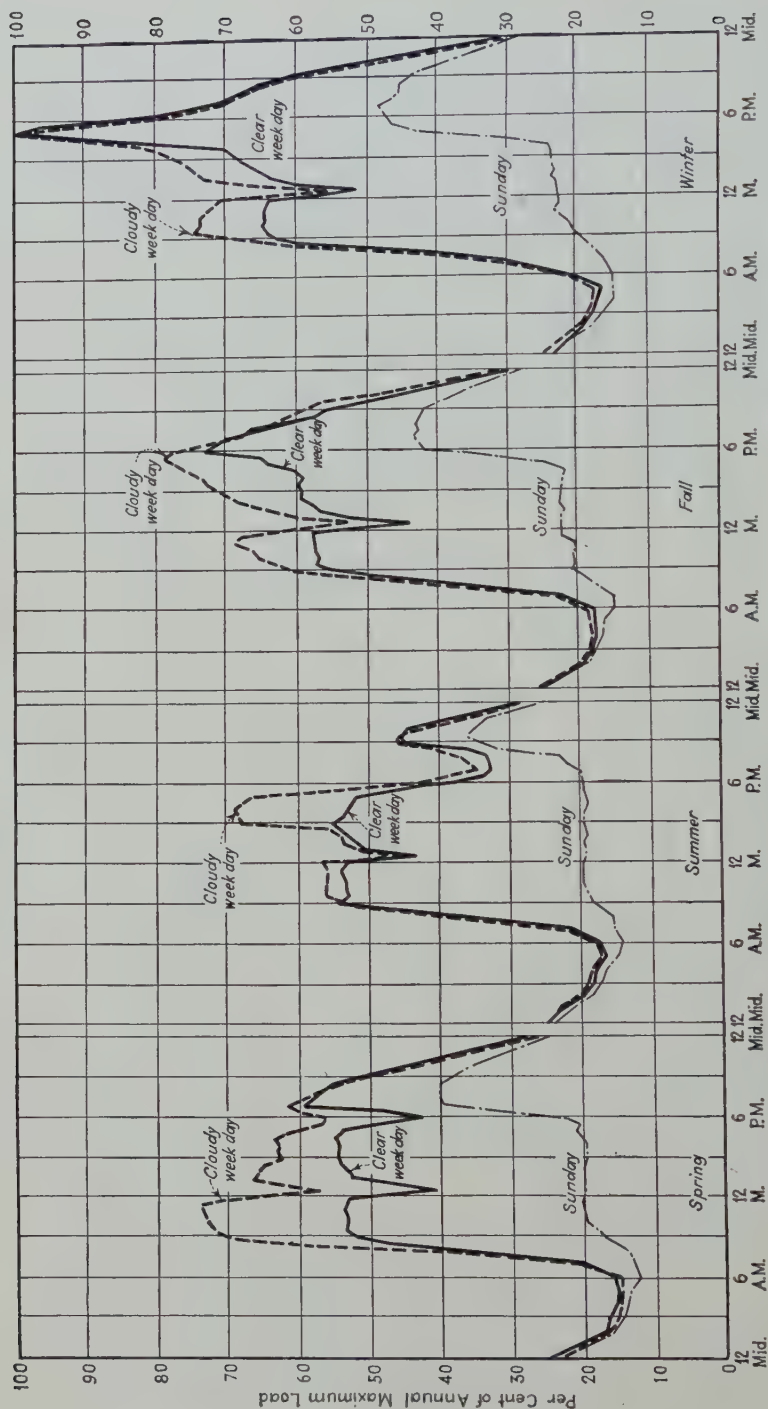


Fig. 282.—The New York Edison and United Companies—typical daily load curves, for the year 1922.

For further use in the study of typical daily load curves, there are shown in Fig. 282 four sets of seasonal curves, each group including curves for a cloudy and clear weekday and a Sunday, for the New York Edison and United Companies for the year 1922, plotted as per cent of annual maximum load. From Fig. 282 has been prepared Table 116, showing some of the characteristics of these load curves, with assumed seasonal period limitations as noted. For the average year thus obtained, the maximum daily load averages 70 per cent of the yearly peak; the average load is 42 per cent of the yearly peak; the average daily load factor is 60 per cent, and 19 per cent of the daily load is above the average load. The last two conditions are closely consistent with Col. 6, Table 54.

Hydroelectric Power in a Large Load System.—If the hydroelectric plant is a unit in a large load system, it may replace a substantial amount of steam capacity.

Thus, consider a hydroelectric plant upon a river with drainage area of 5000 sq. miles with wheel capacity of about 13,000 sec.-ft. and a minimum weekly flow of 1500 sec.-ft., under an effective head of 50 ft. The plant is connected with a power system where the weekly load factor at time of minimum flow is 60 per cent, with characteristics as in Cols. 5-7, Table 54. The average daily load on the system is 3 mill. kw.-hr.

The plant capacity is 60,000 hp., or 42,000 kw., and the 24-hr. power available at the plant during the time of minimum flow is about 7000 hp., or 4900 kw., and its daily available output would be 118,000 kw.-hr., or about 4 per cent of the average daily energy requirements of the system. From Col. 6, Table 55, this would carry the portion of the load curve above about 80 per cent of the peak load. The latter is

$$\frac{3,000,000}{24 \times 0.60} = 210,000 \text{ kw.}$$

The plant could therefore carry 20 per cent of 210,000 kw. upon a 0.2 load factor if used wholly at time of peak load, or 42,000 kw. of capacity provided adequate pondage was available.

The pondage required (Col. 5, Table 55) would be for about 19 hr., requiring a pond capacity of

$$19\frac{1}{2} \times 1500 = 2400 \text{ acre-ft.}$$

With this pondage available the firm power capacity at the plant used in the power systems as described would be 42,000 kw., as compared with only 4900 kw. if no pondage were available. The importance of pondage at the plant is thus again emphasized.

Assuming further that the hydroelectric plant will cost \$125 per horse power of capacity, with an added cost of \$5 per kilowatt-year for transmission, and that it will replace steam power with plant cost of \$100 per kilowatt, fixed operating costs at \$6 per kilowatt of capacity and incre-

ment operating costs of 0.2 ct. per kilowatt-hour. In addition to the daily firm hydroelectric power of 118,000 kw.-hr., or about 43 mill. kw.-hr. yearly, there will be about 60 mill. kw.-hr. yearly of secondary power available for base-load use.

Comparative costs of steam and hydroelectric power would be as follows:

Steam

Fixed charges: $\$100 \times 42,000 \times 0.14 =$	\$ 590,000
Operating cost—fixed: $\$6 \times 42,000 =$	252,000
Operating cost—increment: 103 mill. kw.-hr. $\times 0.002$	<u>206,000</u>
	\$1,048,000

Hydroelectric

Fixed charges: $\$125 \times 60,000 \times 0.10 =$	\$750,000
Operating cost: 103 mill. kw.-hr. $\times 0.0003 =$	31,000
Transmission cost: $\$5.00 \times 35,000 =$	<u>175,000</u>
	\$956,000

In this case the use of the hydroelectric plant for carrying the upper portion of the peak load is somewhat more advantageous.

There would be certain further advantages in the use of hydroelectric power in such a case, in place of equivalent steam power. Thus, with hydroelectric capacity and ample pondage, fewer boilers need be kept hot and fewer steam units warmed up and turned over, either unloaded or inefficiently loaded. A hydroelectric unit, if not completely loaded, can quickly take increase in load and permit the steam units to operate at a more constant loading at or near maximum efficiency. Even when boilers are hot it takes a large modern steam turbine, when starting cold, an hour or more to be warmed up and fully loaded, whereas a hydroelectric unit can be started and picks up load in 1 to 3 min.

Increment Cost of Hydroelectric Plants.—Certain items of the cost of hydroelectric developments which are approximately proportional to the wheel capacity may for convenience be called increment costs. These items include the cost of power house and equipment and to some extent, depending upon the type of development and character of load, the cost of intake and waterway. The cost of dam and reservoir is independent of wheel capacity, while with a concentrated-fall development the cost of intake and short penstock through the dam would vary with it. In the case of a long waterway—canal or penstock where a large forebay or reservoir near the power house is possible—the cost of water-

way may be only slightly affected by the amount of installation, whereas without such a forebay or reservoir the cost may vary with wheel capacity, particularly where the load factor is low.

TABLE 117.—PEAK OR DAY-LOAD PLANTS

Plant	Head, feet	Capacity			Cost per horse power			
		Horse- power	Sec.-ft. per square mile	Flow per cent of time	Power house and equip- ment	Water- way	Dam	Total
Safe Harbor.	53	255,000 ^a	2.0 ^a	25 ^a	\$73	...	\$30	\$103
Lower Fifteen Mile Falls.	170	216,000	8.5	2	24	\$16 ^b	35	75
Davis Bridge	360	60,000	9.6	5	24	48	78	150
Tallulah Falls.....	580	85,000	8.4	5	32	31	31	94

^a Ultimate installation = 510,000 hp., or 4 sec.-ft. per square mile and 3 per cent flow.

^b Including tailrace and intake.

Of the plants listed in Tables 102 and 104, four are used essentially for peak or day loads. These are listed in Table 117 with some of their characteristics and unit costs. They all are (or will ultimately be in the case of the Safe Harbor plant) developed up to a relatively high flow—that available from 2 to 5 per cent of the time. The approximate increment costs per horse power for those plants as constructed are:

Safe Harbor.....	\$73
Fifteen Mile Falls.....	24 + \$16 = \$40
Davis Bridge.....	24 + 48 = 72
Tallulah Falls.....	32 + 31 = 63

In the case of Safe Harbor the cost of \$73 per horse power includes substructure for one future unit and intake structure, tailrace excavation, etc., for the full ultimate capacity; hence, the future increment cost will be materially less than \$73 per horse power.

The type of hydroelectric plant best adapted for low increment cost is that with concentrated fall, where increased capacity affects chiefly power-house and equipment costs, as illustrated by the Safe Harbor and Lower Fifteen Mile Falls plants. Low increment cost of hydroelectric plants has an important bearing upon steam replacement in the comparative economy of the two types of plants. Additional steam capacity, depending upon its character, will cost from \$100 to \$125 per kilowatt, whereas the necessary hydroelectric increment cost to replace steam is often as low as \$50 to \$70 per kilowatt, giving a distinct advantage to the hydroelectric plant under these conditions. Another advantage

which accrues from using hydroelectric plants in a load system is, therefore, that of low increment cost.

Value of a Water Power Site.—A water power site will have a greater or less value, as previously explained, depending upon (1) its natural characteristics, which affect cost of development, and (2) its regional characteristics, which affect the use or market for the power. The value of a water power site may, therefore, vary from nothing, where construction cost would be very high or where too remote from a market, up to an amount which represents the capitalized value of the saving in cost of power afforded by its use at the market that is available.

In valuing a water power site three different methods are used, as follows: (1) real estate method—fixing a price per horse power of capacity with a reasonable assumed wheel installation, based upon the sale price per horse power of water power privileges of similar character in the general vicinity, regarding the water privilege in the nature of real estate; (2) net income method—determining by estimate the net income available from an assumed water power plant at the site, this being the difference between the annual cost of the water power and the prevailing wholesale price for power in the vicinity, and capitalizing this net income at a fairly high rate, such as 10 per cent, to offset the inherent investment risk in such a development; (3) steam power comparison method—making a comparison of cost of water power with that of steam power for the vicinity, capitalizing the difference in favor of the water power, if any, as in (2).

1. *Real Estate Method.*—Theoretically these three methods would give approximately the same result, because the wholesale price for power is usually consistent with the cost of steam power, and the “real estate value,” if a sound one, should also be fundamentally based upon the saving possible by use of the water power.

Practically many water power privileges have been bought and sold as real estate for prices entirely at variance with true economic value, in some cases far below and in other cases far above the fair market value. This method of valuation while simple and easily understood, must, therefore, be used with great care and judgment and should preferably be checked up or tested by one or both of the other two methods outlined above.

Another method of expressing value based upon sale, award, or agreement is “per square mile (of drainage area) per foot of fall,” first suggested by a committee of the New England Water Works Association, which reported in considerable detail upon the value of both undeveloped and developed water powers.¹

In Table 118 are given data of value of undeveloped water power, in part from this committee report and in part from the files and records

¹ *Jour. New Eng. Water Works Assoc.*, March, 1910.

TABLE 118.—VALUE OF UNDEVELOPED, ABANDONED, OR UNUSED WATER POWER PRIVILEGES

Num- ber	Location River	State	Drainage area, square miles		Head, feet	Wheel (or estimated) horse power		Amount paid		Remarks	Date
			Total	Taken		Total	Taken	Per horse power	Per square mile per foot fall		
4	Ten Mile	Conn.	6.3	1.6	16	14.6	3.7	\$ 40.50	\$ 5.86	Cider and saw mill	1908
7	Great Brook	Conn.	16.4	6.44	9.5	22.7	8.9	135.00	19.60	Includes mill site	1901
8	Great Brook	Conn.	16.4	6.44	9.1	21.7	8.5	177.00	25.60		1901
9	Great Brook	Conn.	14.2	6.44	11	22.7	10.3	58.00	8.50		1908
14	Whetstone Brook	Conn.	14.75	0.43	12	25.7	0.75	88.00	12.80	Abandoned and dismantled privilege	1888
39	Kennebec (Tributary)	Me.	200	200	48	(980)	(980)	23.80	2.50	Power not used for years, mill burned down	1901
126	Munn's Brook	Mass.	21	6.4	10	30.5	9.3	53.80	6.98		1900
151	Quackenkill Brook	N. Y.	91.3	17.5	55.1	730	140	42.90	6.23	Storage controlled by privilege below	
152	Quackenkill Brook	N. Y.	91.3	17.5	10.6	140	27	37.00	5.40	Storage controlled by privilege below	
154	Quackenkill Brook	N. Y.	18.1	17.5	74	194	188	7.20	1.04	Storage controlled by privilege below, abandoned privileges	
161	Taughanock Brook	N. Y.	67	67	543	5,300	5300	15.10	2.21		
165	Fish Creek	N. Y.	135	135	273	2,500	2500	2.90	0.20		
169	Tomhannock Creek	N. Y.	75	67	150	1,640	1465	15.00	2.19	Privilege abandoned and plant destroyed	1903
170	Tomhannock Creek	N. Y.	78	67	40	455	390	41.00	5.97	Formerly developed, plant destroyed	
171	Tomhannock Creek	N. Y.	80	67	8	93	78	15.40	2.24	Sale	1913
300	Kennebec	Me.	1300		350	68,000		10	1.43	Sale	1917
301	Kennebec	Me.	3950		32	12,000		33	3.95	Sale	1915
302	Androscoggin	Me.	2800		30	8,400		34	3.40	Sale	1915
303	Androscoggin	Me.	2800		40	12,000		30	3.10	Sale	1916
304	Mendon Stream	Vt.	10	10	43	120	120	22	5.80	Settlement	1921
305	Spicket	N. H.	23		12	40		300	43.50	Sale—abandoned privilege	1921
306	Spicket	N. H.	22		10	40		75	11.30	Sale—abandoned privilege	1921
307	Ipswich	Mass.	43		12	70	30	500	35.00	Settlement—unused privilege	1920
308	Presumpscot	Me.	436		40	3,000		25	4.30	Cost of privilege before development	1903
309	Presumpscot	Me.	443		51	5,000		46	10.20	Cost of privilege before development	1913
310	Presumpscot	Me.	569		28	2,400		104	15.70	Cost of privilege before development	1907

of the writer (Nos. 300 to 310, inclusive). Many of these figures are the result of court decisions in cases where a portion or all of the power was taken or destroyed by municipal water-supply projects, some from agreement and others from actual sales. The wide range in figures, from a nominal amount up to about \$300 per horse power, reflects the great variation in value of undeveloped water power depending on its location, ease of development, and regularity of river flow. In some cases of sales the price paid is, however, higher than actual value and represents what might be called a "nuisance value," the result of a forced transaction.

It has been shown that a cost of water power may be feasible up to even \$300 or more per horse power under some circumstances, under which condition a fairly low cost of development, as \$100 per horse power, would mean a value for the power privilege of \$200 or more per horse power. Ordinarily, however, sales of undeveloped water privileges will run from a nominal amount up to about \$100 per horse power, unless special circumstances prevail.

2. *Net Income Method.*—In the net income method of determining the value of an undeveloped water power privilege, the cost of development must first be estimated as a basis for the determination of fixed charges. This will require a fairly careful preliminary layout and plans, and all probable items of cost should be liberally estimated. Depending upon size of plant and its location with reference to other plants, operating costs will be from 0.05 to 0.15 ct. per kilowatt-hour. In each case a schedule of necessary attendants and cost should be prepared as a basis for this item.

The wholesale price for power in the vicinity must be determined from the best information available, preferably from existing power contracts. The published or filed power rates of the various state public service commissions are helpful in such an investigation. The problem is usually complicated by the fact that a considerable portion of the water power is secondary power, for which there may or may not be a ready market. In such a case it may be desirable to determine the cost of auxiliary steam power sufficient to make all or some considerable portion of the output primary power.

The results of such a study may be summarized as shown by the following example:

EXAMPLE OF NET INCOME METHOD OF DETERMINING THE VALUE OF A WATER POWER PRIVILEGE

Assumed Conditions:

Drainage area = 600 sq. miles; head, 14 ft.; wheel capacity, 800 hp.; plant cost = \$130,000; average yearly output = 2.6 mill. kw.-hr., of which about one-third is primary power; cost of steam auxiliary = 2.0 cts. per kilowatt-hour (including allow-

ance for fixed charges); sale price of primary power = 1.2 cts. per kilowatt-hour, surplus power 0.5 ct.

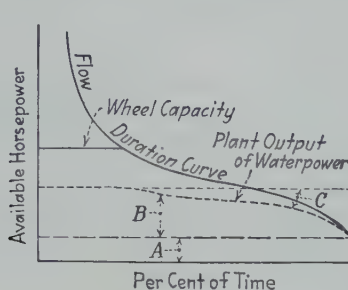


FIG. 283.

Figure 283 shows conditions as assumed where area *A* is primary power, area *B* secondary power, and area *C* auxiliary steam power required to make all power primary.

Case I. Use of Water Power Alone.

Yearly cost:

Fixed charges, 11 per cent of \$130,000.....	\$14,300
Operation.....	4,000

\$18,300

Yearly income:

0.9 mill. kw.-hr. at 1.2 cts.....	\$10,800
1.7 mill. kw.-hr. at 0.5 ct.....	8,500

\$19,300

Net yearly income..... \$ 1,000

Capitalized at 10 per cent = \$10,000 = approximate value of water power, or \$10,000/800 = \$12.50 per horse power.

On this basis the water power is of small value.

Case II. Use of Water Power and Steam Auxiliary.

Yearly cost:

Fixed charges and operation as before.....	\$18,300
0.9 mill. kw.-hr. steam relay at 2.0 cts.....	18,000

\$36,300

Yearly income:

3.5 mill. kw.-hr. at 1.2 cts.....	\$42,000
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\$42,000

Net yearly income..... \$ 5,700

Capitalized at 10 per cent = \$57,000 or \$71 per horse power of wheel capacity.

This is more truly the value of the water privilege and, if added to plant cost, gives about \$233 per horse power of capacity as total value of plant and privilege.

The value of secondary power is broadly speaking represented by fuel saving, for normally it may be assumed that the steam plant is available and its fixed charges not increased by the use of water power. This saving may approach zero at low heads and off-peak hours of the night, as usually some steam power must be kept in use at such times.

During the day, however, especially where pondage permits the water power to carry the peak load, saving may be effected both by water power steam replacement and by permitting a more regular output of steam power.

Secondary power expressed by coal saving would represent from about 0.6 to 0.8 ct. per kilowatt-hour (and in large modern plants as low as 0.2 ct. per kilowatt-hour), depending on coal use and cost per ton. From this viewpoint, depending upon particular conditions and making some allowance for its intermittent character, it might have a sale price of $\frac{1}{5}$ to $\frac{1}{2}$ ct. per kilowatt-hour. The price of secondary power is often assumed at from one-third to one-half of the primary power rate, depending on circumstances.

It is very evident from the preceding example, however, that an auxiliary steam plant is required to develop the full value of a water power development, where much of the power is secondary in nature.

3. Steam Power Comparison Method.—The method of determining the value of an undeveloped water power by comparison with cost of steam power is similar in detail to the net income method previously described, using, however, the capitalized saving in cost, if any, of water over that of steam power, as a basis for the value of the water power. While useful in some cases, the steam power comparison method makes no allowance for other competitive water power as affecting the price for power and is not so direct or conclusive a measure of value as that effected by the net income method.

Value of Developed Water Power.—The value of the water privilege as a developed water power may be determined in the same manner as that described for an undeveloped power. With a completed plant, the fixed and operating costs are, however, known with some certainty and are not a matter of estimate. The risk involved in construction, if any, has also been met, and the plant has the nature of a more stable investment. The rate of capitalization applied to the yearly net income may, therefore, be lower (perhaps 6 to 8 per cent instead of 10 per cent) and a higher value established for the water power than would be warranted before its development.

The value of the plant itself would also be included where the value of the entire development is desired. Depending upon the purpose of the valuation, the plant cost may be the reproduction or present cost new, the first or actual cost, or some amount between these two. Where water power has been taken on account of municipal water supply or other public use, normally a part or all of the plant value has been included in the price paid. Where the power taken is but a fraction of the total power, the plant may still be used, but, with all or a major portion of the power taken, the plant is of no value and its value must be part of the award.

TABLE 119.—VALUE OF DEVELOPED WATER PRIVILEGES

Num- ber	Location		State	Drainage area, square miles		Head, feet	Wheel (or esti- mated) horse power		Amount paid		Remarks	Date
		River		Total	Taken		Total	Taken	Per square horse power	Per foot fall		
16	Great Brook		Conn.	6.44	6.44	10	9.3	9.3	\$750	\$108.60	Small sawmill included in agreement	1901
18	Brigg's Brook		Conn.	2	1.64	51	14.8	12.2	655	95.60	Includes land for dam site; also used as ice privilege	1897
27	French		Conn.	94	94	1.5	20.6	20.6	365	53.20	Cotton mill 10-hr. power	1870
29	West Branch, Naugatuck		Conn.	20	18	7	20.4	18.4	109	15.90	Gristmill	
30	West Branch, Naugatuck		Conn.	21	18	12	36.6	31.5	95	13.90	Sawmill	
31	West Branch, Naugatuck		Conn.	23	18	13	43.5	34	206	29.90	Cutlery factory	
32	West Branch, Naugatuck		Conn.	23.2	18	13	23.6	18.3	65	9.52	Sawmill	
75	Kettle Brook		Mass.	5.02	3.86	22.35	45	34.6	885	242.80	Woolen mills	1895
77	Mass.		Mass.	5.17	3.86	22.35	29	21.6	467	242.80	Satline mill	
51	Beaver Brook		Mass.	2.68	2.6	20.5	8	8	125	18.80	Silk print works 10-hr. power	1900
52	Wachusett reservoir		Mass.	113.4	113.4	15.5	196	196	317	35.20	Yarn mill	1897
54	Sudbury		Mass.	118.23	118.23	28	535	535	540	87.30	Cloth mill	1878
62	East Meadow		Mass.	100	75.2	26	378	284	460	67.20		1896
63	East Meadow		Mass.	7.75	7.75	9	10	10	300	43.00	10-hr. power	1899
68	South Branch, Nashua		Mass.	7.5	7.5	8	8.7	8.7	344	50.00	10-hr. power	1897
79	Sudbury		Mass.	128.8	118.2	8	150	138	254	37.00	Cotton mill	1895
92	Nemasket		Mass.	2.6	2.6	18	6.8	6.8	280	40.50	Includes land	
				59	48.1	10	50	122	61	12.70	Electric lighting plants, 150-hp. steam	1909
107	South Branch, Nashua		Mass.	94.8	94.8	27	617	617	146	35.15	Part of 10.9 sq. miles additional area taken	1896
108	Quinepoxet		Mass.	53.8	53.8	39.7	382	382	136	24.30	White cotton mill	1896
				54.2	54.2						Cotton mills, two developments, 10-hr. power	
118	Branch of Stony Brook		Mass.	4.14	4.14	8	4.8	4.8	350	51.35		1896
122	Nashua		Mass.	98.3	98.3	11	128	128	172	20.70	Cloth mill 10-hr. power	1903
123	Nashua		Mass.	98.3	98.3	11	125	125	176	20.70	Gristmill, intermittent power	1896
124	Nashua		Mass.	98.9	98.9	18.4	178	178	300	29.10	Corset, jeans, quilts, etc.	1888
141	Little Squam Lake		N. H.	56.75	56.75	29	240	240	48	6.08	Gristmill and repair shop, 24-hr. power	1884
142	Newround Lake and River		N. H.	91	91	46	610	610	24	3.58	Sawmill and pulp mill, 24-hr. power	1884
147	Lake Winnepesaukee		N. H.	366	366	12	1640	1640	52	6.75	Cotton and woolen mills 12 and 24-hr. power. Includes land and buildings	1889
148	Smith's and Crooked Ponds		N. H.	428	428	16	1640	1640	52	6.75		1889
400	Pembiscot		N. H.	24	24	18	63	63	95	13.87		1889
401	Spicket		N. H.	5100	5100	19	2860	2860	157	4.65	Sale	1925
402	Ipswich		Mass.	127	127	16	46	46	500	3.75	Sale	1923
403	Otter Creek		Vt.	855	855	35	66	66	6.5	4.20	Agreement	1920
404	Saranac		N. Y.	607	607	9	100	100	150	2.76	Sale	1912

TABLE 120.—COST AND CONDITIONS OF USE OF POWER STORAGE RESERVOIRS

Num- ber	Reservoir or site	River and location	Drainage area, square miles	Area, square miles at high- water level	Maxi- mum draft, feet	Storage capacity, million cubic feet	Head using stor- age, feet	Cost			Date	Remarks or references
								Total	Per million cubic feet	Per million cubic feet per foot of head		
Reservoirs con- structed:												
1	La Loutre	St. Maurice, Que.	3650	300	47	160,000	290	\$ 2,500,000	\$ 15	\$ 0.05	1917	Largest power reservoir
2	Ripogenus	Penobscot, Me.	1410	43.5	50	30,000	180	1,000,000	33	0.18	1918	
3	Kenogami	Saguenay, Que.	1400	23	32	13,000	500	4,000,000	307	0.61	1924	
4	Sacandaga	Hudson, N. Y.	1044	42	31	30,000	473	12,000,000	400	0.85	1930	Hudson River Regulating District
5	Brassua Lake	Kennebec, Me.	675	13.6	30	8,600	197	800,000 ^a	94	0.48		
6	Lake St. Francis	St. Francis, Que.	472	19.7	27	12,200	251	750,000	61	0.24	1916	
7	Azisobos	Androscoggin, Me.	215	10.5	47	9,600	500	1,000,000	105	0.21	1911	
8	Davis Bridge	Deerfield, Vt.	184	3.5	100	5,000	820	2,000,000 ±	400	0.49	1924	Power also developed; one-half cost of dam used as storage cost
9	Indian Lake	Hudson, N. Y.	129	7.8	40	4,600	473	83,600	18	0.04	1896	
10	Somerset	Deerfield, Vt.	30	2.6	85	2,700	1050	800,000	296	0.28	1913	
11		New York	24.4	1.2	75	1,200	550	900,000	750	1.37	1925	
12	Carmel	AuSable, N. Y.	21	1.3	22	508	376	120,000	237	0.63	1926	Arranged for ultimate additional 4 ft. of depth and total storage of 655 mill. cu. ft.
13	Howe, Childs, and Seaver	Minnewawa Brook, N. H.	{ 4.5 1.4 0.5	{ 0.30 0.19 0.07	{ 14 14 18	{ 65 52 20 — 137	242	57,000	416	1.72	1924	Three of group of 10 small reservoirs totaling 373 mill. cu. ft.

1	Reservoirs pro- posed: Oxbow	Raquette, N. Y.	498	17.8	40	15,000	346	1,745,000	116	0.33	1915	New York Conservation Commission, Raquette River Report
2	Portage	Genesee, N. Y.	950	13.5	90	11,250	264	4,520,000	402	1.52	1908	New York State Water Sup- ply Commission, Fifth Re- port Upper 19 ft. of storage capacity reserved for flood protection
3	Hiram	Saco, Me.	800	35	35	10,000	207	957,000	96	0.46	1918	Maine Public Utility Com- mission, Water Report
4		New York	118	1.7	120	1,600	550	1,600,000	1000	1.82	1926	

^a Not including land and flowage rights.

Much information on prices paid for developed water powers is given in the committee report of the New England Water Works Association already referred to, and data for some of these plants—where 75 per cent or more of the power was taken and, hence, the plant substantially destroyed and included in the award—are appended in Table 119, where are also included some data (Nos. 400 to 404, inclusive) from the files of the writer.

As will be expected, the value per horse power shows great variation—from about \$25 to nearly \$900 per horse power for the plants listed. Considering, however, the four highest:

Number	Horse power	Price per horse power	Remarks
75	35	\$885	Woolen mill (Kettle Brook cases)
16	9	750	Small sawmill included
18	12	655	Amount includes land for dam site; also used as an ice privilege
52	535	540	Cloth mill

Of the above, the first three include value in addition to that of the developed water power. Actually, therefore, the upper limit for developed water power in the table is in the vicinity of \$500 to \$600. Similarly, inspection of the very low values, three of which are respectively \$24, \$48, and \$52, shows that these are all sales of plants in New Hampshire and probably considerably undervalued.

Giving little weight to extreme values in Table 119, therefore, the range in value of developed water power shown varies from about \$100 to \$500 per horse power, which is fairly consistent with the conclusions previously reached. The average for the whole table is about \$250 per horse power.

Cost of Storage Reservoirs.—Numerous reservoirs have been planned for storage of water to be used for power and many constructed. In Table 120 are given data of cost and conditions under which various power-storage reservoirs are used, with similar data for a number projected but not yet built.

Most of these are large reservoirs with a relatively low cost per million cubic feet of storage. The "head using storage" is that now developed on the river below the reservoir, limited in all cases of constructed reservoirs to the developments which contribute to the cost of storage. Thus, for the Azischohos reservoir the head of 500 ft. represents the total head at plants of the four companies which participated in the cost of this storage, while there are some plants on the river below the reservoir which do not share in this cost (about 135 ft. total head in the case of Azischohos reservoir).

In the list of proposed reservoirs the total developed head below the reservoir is given.

With all the reservoirs and sites listed, there are power sites as yet undeveloped, in some cases a large aggregate head, which will eventually benefit from the storage. Thus, over 300 ft. of undeveloped head is found on Androscoggin River below the Aziscohos reservoir.

In the case of Davis Bridge reservoir in Table 120, as the project is one combining power and storage, it is not possible exactly to state the cost of storage. It has been assumed here that the cost of storage is represented by approximately one-half the cost of the dam and its tunnel spillway (including also a portion of the overhead charges on the job). The relative cost of this power and storage may also be estimated in the following manner:

The yearly average output from the Davis Bridge plant and from the use of its stored water at plants below is about 136 mill. kw.-hr., the cost of which is (see details of cost, Chap. XIII):

Fixed charges:

\$7,000,000 at 10 per cent.....	\$700,000
3,000,000 at 6 per cent.....	180,000
	\$880,000
Operation.....	36,000
Total yearly cost.....	\$916,000

$$\frac{\$916,000}{136,000,000} = 0.67 \text{ ct. per kilowatt-hour}$$

which is the approximate cost at switchboard of Davis Bridge and the other plants below. This is of course all primary power, and its cost indicates the value of this reservoir when its stored water is used through head aggregating 820 ft.

The small reservoirs on Minnewawa Brook in Table 120 are three of a system of 10 reservoirs which total 373 mill. cu. ft. of capacity. The cost of the other seven reservoirs was relatively small, as they are ponds or lakes which have been raised in level. The cost of storage for the system is, therefore, much less than for the three reservoirs alone.

Cost of Reservoirs in Vermont.—A comprehensive study of power reservoirs in Vermont made by the author¹ to determine the possibilities of flood control gave the following summarized results:

Upon 21 rivers, with a total drainage area of 5470 sq. miles, 85 reservoir sites with storage capacity totaling 83 bill. cu. ft. below spillway level and 34 bill. cu. ft. above spillway level were selected, which would provide

¹ *Reports of Advisory Committee of Engineers on Flood Control, Vermont, and of consulting engineer to committee, 1928 and 1930.*

about 15 mill. cu. ft. per square mile of storage below spillway level and about 6 mill. cu. ft. above spillway level.

The estimated cost of these 85 storage projects totaled about \$83,000,-000 or an average of about \$1000 per million cubic feet of capacity below spillway level, ranging from about \$300 to about \$4000.

The cost of the better 27 of these projects, with 44 bill. cu. ft. of storage, or more than half the total, averaged \$650 and ranged from \$270 to \$1000 per million cubic feet.

The cost per million cubic feet per foot of head through which stored water would be used was for the 85 projects \$2.76—ranging from about \$0.50 to \$10. For 26 of the better projects, totaling 28 bill. cu. ft. of storage, the cost averaged about \$1.56—ranging from about \$0.50 to \$2.40 per million cubic feet per foot of head.

These figures show well the range of cost of power-storage reservoirs in northern New England.

Methods of Valuation of Storage Reservoirs. *Coal-saving Method.* This method of valuating storage consists of determining the yearly value of coal saved corresponding to the water power generated by water in the reservoir and the available head through which it is used, and capitalizing this yearly saving to determine the available limit of expenditure for storage. It is only a rough method of analysis and fails to take into account the gain in primary power resulting from the use of stored water.

Net Income Method.—The best method of valuation of storage consists in estimating the yearly income from power with and without storage and by comparison the resulting gain, if any, by use of storage.

Thus, considering the results of storage on the Saco River, as shown in Chap. III, Fig. 49, and using 0.3 and 0.8 ct. per kilowatt-hour, respectively, as the value of secondary and primary power, the comparative yearly output and its value with and without storage are as follows, where a head of 207 ft. is available:

Without storage:

Primary power, 69 mill. kw.-hr. at 0.8 ct.	\$ 552,000
Secondary power, 161 mill. kw.-hr. at 0.3 ct.	483,000
	\$1,035,000

With storage:

Primary power, 200 mill. kw.-hr. at 0.8 ct.	\$1,600,000
Secondary power, 65 mill. kw.-hr. at 0.3 ct.	195,000
	\$1,795,000
Gain due to storage, yearly	\$ 760,000

Storage per square mile in this case is about 11 mill. cu. ft. for which the use factor would be about 0.95 (see page 151) and the yearly gain $\$760,000 \times 0.95 = \$720,000$. Capitalized at 10 per cent this is \$7,200,-

000 or $\frac{\$7,200,000}{14,000,000,000 \times 207} = \2.50 per million cubic feet of fall, as the limit of expenditure for storage.

The general effect of storage in increasing primary power may also be determined from Fig. 41, Chap. III as shown in Table 121.

TABLE 121.—EFFECT OF STORAGE ON PRIMARY POWER

Storage capacity		Flow as ratio to yearly mean		Ratio Col. 4 Col. 1
As ratio to yearly mean flow	Million cubic feet per square mile (mean flow = 1.60 sec.-ft. per square mile	Total	Increase over minimum	
(1)	(2)	(3)	(4)	(5)
0.0	0	0.24	0.00	
0.1	5	0.46	0.22	2.2
0.2	10	0.61	0.37	1.85
0.3	15	0.70	0.46	1.53
0.4	20	0.72	0.48	1.2

Thus, as shown in Col. 5 for a typical river in the northeastern United States, the effect of storage up to 5 mill. cu. ft. per square mile is to increase the minimum flow about two and two-tenths times the storage volume. As storage per square mile increases, this ratio of increase lessens and becomes unity for high amounts of storage. One million cubic feet of storage (23 acre-ft.) per square mile of drainage area corresponds to a daily flow of 11.5 sec.-ft. which for 1-ft. head is about $\frac{3}{4}$ kilowatt or 18 kw.-hr. per day.

For moderate storage—up to, say about, 7.5 mill. cu. ft. per square mile—the effective increase in primary power would be about $2 \times 18 = 36$ kw. per square mile per foot of fall. Changing power from secondary to primary is worth about $0.75 - 0.15 = 0.60$ ct. per kilowatt-hour or 21.6 cts. for 36 kw.-hr. An expenditure of \$2.40 will at 9 per cent return this amount yearly, thus fixing the maximum allowable expenditure for moderate storage at \$2.40 per million cubic feet per foot of fall.

For a high degree of storage only about one-half of this or \$1.20 per million cubic feet per foot of fall would be warranted. This analysis is conservative, however, in that it does not take into account the use of hydroelectric power in peak loads where the primary power is worth more than $\frac{3}{4}$ ct. per kilowatt-hour as assumed above.

In general, approximate limits in economical expenditure for storage may be taken as about \$2.50 per million cubic feet per foot of fall for moderate degrees of storage—up to, say about, 7.5 mill. cu. ft. per square

mile and about \$1 per million cubic feet per foot of fall for large proportionate storage.

Annual Cost of Storage.—The annual cost of storage will consist of fixed and operating charges. The fixed charges are computed as for power developments, but the total percentage allowance will be somewhat less, as depreciation and maintenance costs will be less, owing to a relatively small proportion of equipment cost. Commonly a total of 8 to 9 per cent yearly will cover fixed charges.

Operating costs consist chiefly of cost of attendance, keeping records, etc., appurtenant to storage use. The author showed¹ that for storage reservoirs in Maine an annual cost allowance of about 50 cts. per million cubic feet of storage capacity for operation would be ample. These reservoirs are owned and controlled by a few companies, however, under agreement, and in other cases the cost of operation may be as much as \$1 and \$2 per million cubic feet yearly.

The estimated annual cost of the Sacandaga reservoir, for which the estimated first cost and total cost as constructed were given in detail in Chap. V, pages 270–271 is as follows (revised to 1932):

SACANDAGA RESERVOIR—ESTIMATED ANNUAL CHARGES FOR MAINTENANCE AND OPERATION

Total Cost = \$12,000,000

1. Interest at $4\frac{1}{4}$ per cent and yearly retirement of 50-year bonds (one-fiftieth each year). This will require a first yearly payment of about \$750,000 and a last yearly payment of about \$240,000; the average yearly payment will be about.....	\$495,000
2. Depreciation and repairs, 3 per cent on \$1,000,000.....	30,000
3. Taxes.....	50,000
4. Operating expenses, including superintendence, labor, rent, supplies, engineering, and legal expenses incident to operating reservoir and collecting assessments.....	50,000
Total.....	\$625,000

As will be noted, the estimated yearly operating expenses as given above are about \$50,000, for a reservoir capacity of 30 bill. cu. ft. or about \$1.67 per million cubic feet yearly. The total yearly cost of \$625,000 represents about \$21 per million cubic feet of capacity.

This reservoir increases the continuous or primary power of the whole river from 34,400 to 132,500 or 98,100 hp., costing yearly $\frac{\$625,000}{98,100}$ = \$6.30 per horse power year of primary power or about 0.10 ct. per kilowatt-hour at switchboard.

At existing plants on the Hudson River 35,700 hp., practically all firm power, is added at a cost of \$17.50 per horse-power-year or 0.28 ct. per kilowatt-hour.

¹ BARROWS, H. K., "Cost of Power in the State of Maine," Public Utilities Commission of State of Maine, pp. 396–397, Nov. 27, 1918.

The assessment on present developed power (473 ft. and 210,000 hp.) is about

$$\frac{\$625,000}{210,000} = \$3.00 \text{ per horsepower of capacity.}$$

If an additional fall of 192 ft., just below the proposed reservoir, is included, the total assessed horse power would be 227,000 and the assessment about \$2.80 per horse power. The costs given above are storage costs only and do not include fixed or operating costs of power plants themselves.

Storage development, obtained, as in this case, at reasonable cost, and where many plants can use the stored water, is very obviously desirable, both financially and as a conservation measure.

Pumped-storage Hydroelectric Plants.—The general requirements for pumped-storage hydroelectric plants were given in Chap. III, page 178. Following is a description of two plants of the kind in this country.

*Rocky River Plant.*¹—Rocky River is a small tributary of the Housatonic River entering it near New Milford, Conn. A hydraulic-fill earth dam 112 ft. high, with timber core wall, was built upon Rocky River near its mouth. A canal 3200 ft. long, followed by a 15-ft. steel penstock 1900 ft. long, runs to a power house on Housatonic River containing one 24,000-kw. power unit under 230-ft. head (with provision for a second unit) and two 8150-hp. pumping units, each of 250 sec.-ft. capacity. The dam creates a reservoir of 6 bill. cu. ft. capacity or about 65 in. depth upon the drainage area of 40 sq. miles. Pumping is from the Housatonic River, with a drainage area of 1037 sq. miles. This plant was built in 1926–1927 and added about 40,000 kw. of firm capacity and 80 mill. kw.-hr. yearly to the system, which included two run of stream plants. Stored water is used under a head of 71 ft., in addition to the 230-ft. head at the Rocky River development, at Stevenson on Housatonic River, below Rocky River.

*Lake Lamoka Plant*² *on Keuka Lake, New York.*—This is unique as a combination of the use of hydroelectric power and power from a natural gas field under the hydroelectric property. A capacity of 6600 hp. in four gas-engine units and at present a 2500-hp. hydroelectric unit and one 8000-g.p.m. pump are in use. The hydroelectric and gas-engine units operate in parallel on peak loads, and during off-peak periods the gas-engine units supply power for pumping water from Keuka Lake into Lake Lamoka, the hydroelectric-plant reservoir.

Economics of Pumped-storage Plants.—The essential object of the pumped-storage plant is to improve the load factor of the base-load

¹ AMBERG, "Rocky River Hydroelectric Development," *Jour. A.I.E.E.*, 1928, pp. 1100–1108.

² *Power*, January, 1934, p. 24.

plant and therefore lessen its required capacity. The comparative economy of a pumped-storage plant and of steam power may be determined by their comparison as shown in the following example:

Assume a system load curve as in Table 54 with a 60 per cent load factor which has capacity to carry a peak load of 200,000 kw., all steam power. It is desired to provide for an additional 50,000 kw. of peak capacity with the same load factor. A new steam plant will cost \$100 per kilowatt. The general situation is such that a pumped-storage plant is practicable at a cost of \$100 per kilowatt and it is desired to determine whether this will be more economical than a steam plant of the same capacity. The steam plant is in both cases to carry all the additional off-peak load.

The 50,000 kw. carried as the peak of the load will be $\frac{50,000}{250,000} = 0.20$ of the peak load and will carry the upper 80 per cent of the load. For a 60 per cent load factor (Col. 6, Table 54) this will include 4.0 per cent of the average daily energy, or

$$0.04 \times 250,000 \times 24 \times 0.60 = 144,000 \text{ kw.-hr. daily}$$

This would be about 55 mill. kw.-hr. yearly and would be supplied upon a capacity factor of about 12 per cent.

Because of generation and pumping losses, etc., the pumped-storage plant must be supplied with $\frac{1}{0.85 \times 0.94 \times 0.80} = 1.57$ or, say, 1.6 times 144,000 kw.-hr. daily, or about 230,000 kw.-hrs. daily, or 84 mill. kw.-hr. yearly.

Cost of Steam Power.

Plant cost = 50,000 kw. at \$100.....	\$5,000,000
Yearly cost:	
Fixed charges, 12 per cent of \$5,000,000.....	600,000
Operating cost:	
Fixed, \$5 × 50,000 kw.....	250,000
Increment, 55 mill. kw.-hr. × 0.20 ct.....	110,000
Total.....	\$960,000
or 1.74 ct. per kilowatt-hour	

Cost of Pumped-storage Power.

Plant cost = 50,000 kw. at \$100.....	\$5,000,000
Yearly cost:	
Incremental steam for pumping, 0.20 ct. × 84 mill. kw.-hr.....	168,000
Hydroelectric plant, fixed charges, 10 per cent of \$5,000,000.....	500,000

Hydroelectric-plant operation, 84 mill. kw.-hr. at 0.06 ct.	50,000
Total:	<u>\$718,000</u>

or 1.30 ct. per kilowatt-hour

Difference between yearly cost of steam power and of pumped-storage operation, in favor of the latter	\$242,000
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Hence the pumped storage plant at \$100 per kilowatt would be decidedly advantageous.

References

1. FREEMAN, W. W. K., "Pumped-storage Hydroelectric Plants," *Trans. A.S.C.E.* 1930, pp. 884-935.
2. ANNET, "Pumped-storage Hydroelectric Plants," *Power*, January, 1934, pp. 20-24. This contains a tabulation of all such developments in the world, totaling 64, with their principal features. Germany has 24, Italy 16, and Switzerland eight plants.
3. MAAS, A., "Hydraulic Power Storage Plants," Escher Wyss Engineering Works, Zurich, Switzerland, 1933. This deals with methods of operation and descriptions of 15 plants, mostly in Germany.

Redevelopment of Water Powers.—In the older portions of the country it is sometimes possible to redevelop an existing water power plant with resulting financial gain. Many of the older plants were laid out on a divided-fall basis, where by modern practice a single fall could be used. In the last 25 years a distinct advance in types and efficiency of turbines has been made, so that even replacing old wheels with new may give an added efficiency of 15 to 20 per cent or more.

Thus, the replacement units at Vernon (see Fig. 177) of the single-runner type give an efficiency of about 87 per cent as compared with about 70 per cent for the old three-runner units. For 1200-sec.-ft. use under 34-ft. head, this is a gain of about 800 hp. The cost of these new wheels was about \$40,000 (4200 hp. at \$9.50 per horse power) and the wheel-unit set cost about \$70,000 (or \$16.60 per horse power), using the old generator (previously rewound). For the gain of 800 hp. the cost was about \$88 per horse power, or approximately \$7000 per year for about 2.25 mill. kw.-hr. assuming a wheel unit to run half the time, or about 0.3 ct. per kilowatt-hour, indicating the advantage of this replacement. The old wheel units also had very high maintenance expenses. The gate mechanism of the new unit is all exposed and accessible for lubrication and repairs, instead of being under water. Leakage when the unit is shut down is also reduced from about 50 sec.-ft. for the old units to less than 5 sec.-ft. with the new. Gate links if broken can be replaced imme-

diately instead of waiting for months, as formerly. A further substantial saving in operation will, therefore, result with the new single-runner units.

Where a good market for power exists, any increase in wheel efficiency adds practically in direct ratio to plant income, and it is, therefore, of vital importance to see that proper wheel efficiency is being obtained and, when necessary, to replace inefficient units.

As already stated, the development at Cohoes, N. Y., is an example of a redevelopment in a single fall of an old project, and there are many other sites where similar action might be taken except for the complications of old leases and divided ownership of the fall. As water power becomes more fully developed, however, more of these older projects will in all likelihood be redeveloped and made more efficient.

Cost-index Numbers.—The cost-index numbers used as a basis for readjustment of cost data to a common basis are those of the *Engineering News-Record*.¹ They are based upon average production of materials and labor for the years 1913, 1916, and 1919, with prices as of 1913, as follows:

BASIS FOR COST INDEX—AVERAGE PRODUCTION FOR THE YEARS 1913, 1916, AND 1919	
1. 33 mill. tons steel at \$30.....	\$ 990,000,000
2. 90 mill. bbl. cement at \$1.19.....	107,000,000
3. 42 mill. M ft.b.m. lumber at \$28.50.....	1,197,000,000
4. 1200 mill. man-days (8 hr.) at \$1.52.....	1,822,000,000
	\$4,116,000,000

The prices for materials, which are those that would be commonly used in construction, are those for 1913 as follows:

1. Steel, structural steel, Pittsburgh base.
2. Cement, net, f.o.b. Chicago.
3. Lumber, southern pine, at New York.
4. Labor, common labor, average for the United States.

The basis for the index number of 100 for the year 1913 is supposed to be consistent with a typical piece of construction and was derived in the following manner:

BASIS FOR INDEX NUMBER OF 100 FOR THE BASE YEAR, 1913	
2500 lb. structural steel at 1.5 cts.	\$37.50
6 bbl. cement at \$1.19.....	7.14
600 ft.b.m. lumber at \$28.50 per M.....	17.10
200 man-hr. at \$0.19.....	38.00
	\$99.74

These cost-index numbers are published in the first issue of each month of *Engineering News-Record*, with occasional summaries. Their values by months for the period 1914–1933, inclusive, are given in Table 122.

¹ *Eng. News-Record*, July 21, 1921, p. 127.

TABLE 122.—COST-INDEX NUMBERS, 1914-1933, INCLUSIVE

	1914	1915	1916*	1917	1918	1919	1920	1921	1922	1923	1924	1925
Jan.....	89.22	87.12	112.66	167.75	184.51	198.05	206.55	230.87	168.72	191.70	217.90	210.40
Feb.....	89.22	87.36	116.66	167.75	184.51	201.50	225.10	230.67	168.72	197.40	220.30	209.70
Mar.....	88.97	87.66	125.16	175.66	184.51	198.05	240.85	224.27	162.04	205.25	224.70	210.20
Apr.....	87.97	90.16	129.51	183.41	186.01	191.25	265.20	213.07	164.72	213.50	221.60	209.55
May.....	88.62	90.16	131.06	187.61	186.01	191.25	268.90	210.82	164.62	216.70	222.38	207.20
June.....	87.87	88.96	128.71	199.26	186.31	191.85	273.80	209.82	166.62	220.70	216.85	204.60
July.....	88.12	90.51	127.96	204.01	188.65	193.65	265.65	203.82	169.70	222.10	214.40	204.60
Aug.....	90.11	91.76	128.76	198.41	193.85	196.65	252.00	193.47	173.40	221.50	213.15	204.60
Sept.....	90.36	93.01	131.76	190.31	193.85	202.85	255.20	188.27	185.00	221.50	211.25	202.10
Oct.....	88.51	96.16	135.11	167.11	193.85	202.25	255.20	182.57	188.60	220.30	207.55	205.35
Nov.....	87.26	101.06	138.16	166.51	193.55	206.85	255.32	166.32	188.60	220.90	205.70	205.95
Dec.....	86.51	107.06	149.41	167.11	194.75	206.85	251.62	167.82	192.60	217.30	208.58	205.95
Averages.	88.56	92.58	129.58	181.24	189.20	198.42	251.28	201.81	174.45	214.07	215.36	206.68

	1926	1927	1928	1929	1930	1931	1932	1933	1934
Jan.....	207.15	211.50	203.90	209.40	208.96	194.48	162.48	159.30	191.26
Feb.....	206.55	210.15	204.65	210.40	206.46	196.61	161.82	159.30	194.04
Mar.....	207.65	208.80	204.65	207.78	206.80	194.51	157.24	158.44	194.04
Apr.....	207.05	209.00	206.40	203.40	207.12	191.63	153.12	160.16	195.86
May.....	207.30	206.80	207.00	205.15	205.86	189.33	152.78	164.39	199.61
June.....	204.80	205.55	206.15	205.65	203.36	187.23	152.20	163.41	199.61
July.....	207.80	203.68	206.65	204.77	200.95	174.37	153.36	165.50	
Aug.....	208.30	205.50	207.29	205.91	200.95	171.38	156.80	167.00	
Sept.....	208.30	203.60	207.29	207.57	199.58	171.40	157.96	175.78	
Oct.....	209.80	204.40	207.71	206.32	198.72	169.78	159.16	187.74	
Nov.....	210.80	201.98	209.46	208.46	198.54	169.28	158.20	190.14	
Dec.....	210.80	203.90	210.16	209.46	196.86	166.23	158.46	192.14	
Averages...	208.03	206.24	206.78	207.02	202.85	181.35	156.97	170.28	

Yearly averages prior to 1914: 1903—93.90; 1904—87.40; 1905—90.55; 1906—95.10; 1907—100.55; 1908—97.20; 1909—90.92; 1910—96.33; 1911—93.43; 1912—90.70; 1913—100.00.

* 1916 figures revised in accordance with survey of labor conditions in that year.

CHAPTER XIII

REPORTS AND PLANT DESCRIPTIONS

Hydroelectric Development Outline.—One of the best methods by which the student may become familiar with methods of development and details of design of hydroelectric plants is to visit and carefully inspect and study some typical modern installations, taking notes in detail, supplementing with photographs, and preparing a report upon each plant visited. (Photostat copies of blueprint plans, usually made one-half the size of the original prints, can readily be made for this purpose.)

As an aid in obtaining this information and preparation of the report, a Hydroelectric Development Outline (see Appendix A) has been prepared to be used as far as practicable. Not always however, can complete data be obtained, especially those of cost.

This outline will also be of service as a guide to the engineer in making an inventory and plant appraisal.

Reports.—One of the frequent duties of the hydraulic engineer is to make reports upon water power projects. These may include reports upon:

1. The feasibility and desirability of a proposed water power project.
2. The appraisal and valuation of an existing development, including plant and water rights.
3. The valuation of a water power privilege (or often the damages incurred by the taking of a whole or part for other purposes).
4. Problems relating to the use of water at adjacent plants on the same stream.
5. Backwater and flowage effects, particularly with new projects or where changes in conditions of use of a dam are made.

In preparing such engineering reports, a careful and detailed outline should first be made in logical form and followed in the body of the report. In order that the reader may quickly ascertain the scope and principal results of the work, the report, unless very brief, should begin with a "summary and conclusions in brief" (written as the final step in preparation) where, in concise form, are summarized its purpose and conclusions.

The body of the report should be arranged so that it may readily be followed. If details in the form of tables, etc., are required as a basis for the report, these may usually best be placed in appendix form at the end, so as not to interrupt the continuity of the report, with data used as a means for its preparation.

A complete report upon a proposed water power project may involve much field work of surveys and often subsurface explorations in the form of borings. The report will be accompanied by the plans made from the surveys, as well as such detailed preliminary plans as are necessary to outline the project. The plans should, when possible, always be arranged in sets, with size of sheet not too large to be readily and conveniently looked over.

A report should be explicit and only in such detail as is required to show clearly the basis for its conclusions, particularly avoiding "padding" with unnecessary material.

An outline of a preliminary report upon a water power project is given in Appendix B, with complete text of the "summary and conclusions in brief." This is a report prepared by the author, but with names, localities, etc., changed from the original.

Plant Descriptions and Plans.—In Appendix C are given descriptions accompanied by selected plans giving some of the more interesting features of seven hydroelectric developments in various parts of the country, as follows:

Plant	Location	Capacity, horse power	Head, feet	Date built
Davis Bridge (Harriman)	Deerfield River, Vermont	60,000	365	1923-1925
Searsburg	Deerfield River, Vermont	6,200	230	1921
Marlboro	Minnewawa Brook, New Hampshire	2,500	270	1923
Turners Falls	Connecticut River, Massa- chusetts	70,000	60	1914
Crescent	Mohawk River, New York	8,000	26	1925
Bartlett's Ferry . . .	Chattahoochee River, Georgia	44,000	112	1925
Mystic Lake	Montana Power Company, Montana	15,000	1050	1925

The descriptions are only in brief outline, except for the Davis Bridge development, which is in detail, following the arrangement of the outline in Appendix A. In the case of the Marlboro and Turners Falls plants some detailed costs are also given.

These are representative and typical modern plants illustrating practically all the various types and covering a wide range of head and capacity. The general layout should be studied and analyzed in each case and the plant features compared with practice and design as previously described in the text.

APPENDIX A

HYDROELECTRIC DEVELOPMENT—OUTLINE

I. General.

1. Location, river and town, drainage area.
2. Owner.
3. Date built.
4. Manner of development.
5. Capacity, number, and type of units, head, and speed
6. Use of power.
7. Annual output, kilowatt-hours, average, maximum, and minimum.
8. Cost, total and per horse power of capacity.
9. Special features or conditions.
10. References.

II. Dam.

A. Location and site:

1. River and locality, drainage area.
2. Character of foundation materials.
3. Purpose.

B. Main features:

1. Type.
2. Dimensions:
 - a. Length, bulkhead, spillway, and wing sections.
 - b. Elevations, top and bottom.
 - c. Sections, cross and longitudinal.
3. Materials.
4. Spillway.

C. Accessories:

1. Waste or flood gates: number, kind, size, arrangement, operating mechanism.
2. Flashboards, kind, dimensions, and details.
3. Logway: arrangement, dimensions, and details.
4. Fishway: arrangement, dimensions, and details.
5. Other features.

D. Flood capacity of spillway and gates:

1. Amount, second-feet and second-feet per square mile.
2. Comparison with floods expected.

E. Pondage or storage, area acres, depth feet, and volume second-foot days.

F. Additional notes.

III. Head works.

A. Gates:

1. Number, size, kind, and operating mechanism.
2. Area and velocity at full plant capacity.

B. Racks:

1. Area and arrangement of rackway, supports.
2. Dimensions and spacing of bars.
3. Method of cleaning, disposal of refuse.

C. Gate house: dimensions, arrangement, materials.

D. Other features.

IV. Canal.

A. Location and site:

1. Length, plan, and profile.

2. Type, character of site.

B. Main features:

1. Velocity, slope, total head loss.

C. Accessories:

1. Waste gates.

2. Spillway.

3. Other features.

D. Forebay, size and capacity.

E. Gates or racks at end of canal (description as in III).

V. Penstock.

A. Location and type:

1. Length, plan, and profile.

2. Type, character of site.

B. Main features:

1. Dimensions, sections, details.

2. Cradles, details.

3. Velocity, slope, total head loss.

C. Accessories:

1. Surge tank: kind, dimensions, location.

2. Relief or air valves.

3. Vent pipe.

4. Gates.

D. Other features.

VI. Power house.

A. Location, site, and general arrangement.

B. Substructure:

1. Dimensions and arrangement.

2. Wheel and generator settings and draft tubes.

3. Tailrace.

C. Superstructure:

1. Dimensions and kind of construction, sections.

2. Floor layout.

3. Structural details, walls, and roof.

D. Equipment:

1. Turbines:

a. Number, make, size, type.

b. Horse power, head, revolutions per minute.

c. Characteristics and efficiency.

d. Governor, type and capacity.

2. Generators:

a. Number, make, type.

b. Kilowatts, revolutions per minute, phase, cycles, volts.

c. Ventilation.

3. Exciters: number, capacity, type.

4. Switchboard:

a. Arrangement, type, panels, instruments.

b. Wiring.

c. Oil switches.

5. Transformers: number, capacity, type, arrangement.
6. Lightning arresters: arrangement, type.
7. High-tension connections, outdoor yard.
8. Other features, crane, etc.

VII. Costs.

A. Construction:

1. Dam (and reservoir), gate house.
2. Head works, equipment.
3. Waterway:
 - a. Canal.
 - b. Penstock.
4. Power House.
 - a. Substructure.
 - b. Superstructure.
 - c. Equipment:
 - (1) Hydraulic.
 - (2) Electrical.
 - (3) Miscellaneous.

5. Tailrace.

6. Total.

B. Other costs:

1. Land and water rights.
2. Interest during construction.
3. Miscellaneous.
4. Total.

C. Total cost.

D. Cost per horse power of capacity.

VIII. Power output.

A. Use of power.

B. Load curve and load factor and character of load, utilization factor.

C. Yearly output, kilowatt-hours. Average, maximum, and minimum.

D. Operating force, schedule and pay roll.

E. Cost of power at switchboard:

1. Fixed charges.
2. Operating costs.
3. Total.

IX. Transmission.

A. Length and location of lines.

B. Type of lines, towers or poles, conductors.

C. Capacity, voltage, losses.

D. Costs:

1. Construction.
2. Maintenance and operation.
3. Per kilowatt-hour, total.

APPENDIX B

REPORT UPON RED RIVER POWER DEVELOPMENTS

Submitted to A. B. Chalmers, Vice-president, Westwood Lumber Company, May 1, 1915

E. W. HOLTWOOD & Co.
Consulting Engineers
12 Blank Street
New York

May 1, 1915

Mr. A. B. Chalmers, Vice-president,
Westwood Lumber Company,
126 B. Street,
New York City.

Dear Sir:

In accordance with your directions we have made surveys and investigations of Red River from Bancroft to Granger, N. C., to determine the value of the water rights owned by your company and the probable method of power development, if worth while, in order that you may retain all land necessary for such purposes in disposing of certain of your land holdings to the Eastland Paper Company, and submit the following report upon the matter.

SUMMARY AND CONCLUSIONS IN BRIEF

Power Sites.—Red River between Bancroft and the Granger-Casper line falls about 730 ft. (from an elevation of 1200 to 470) in a distance of 10 miles. The fall is fairly evenly distributed, but concentrated falls of 10 to 15 ft. or more over ledge outcrop occur at the Upper Falls, the Lower Falls, and near Station 65 of the river survey (see sheet 2031.3).

At Bancroft opportunity exists to construct a large storage reservoir. An earth-fill dam about 2800 ft. long will be required for such a reservoir, but the situation is favorable to the use of the hydraulic-fill method in construction, so that the cost will not be excessive.

Method of Development.—The studies made indicate that a large reservoir at Bancroft is most economical, with high-water level at about an elevation of 1260 and a capacity of about 3000 mill. cu. ft. The reservoir dam can also be used for power development.

Six different studies of power development (called *A* to *F*, inclusive, on the plan) have been made, all of which contemplate the use of nearly the entire available fall in Red River. Five plants are required in four of the studies, and four plants in the two other studies.

In all cases the plant near the reservoir (No. 1) and the two in the lower part of the river (No. 4 and No. 5) are included. The variations in method of development occur between and near the Upper and Lower Falls, as will be noted by reference to the set of plans accompanying this report.

The total gross head developed will be from 691 to 722 ft. (depending upon the method considered); the net head from 654 to 681 ft. The total horse power of wheels required will be from 17,750 to 16,600.

Available Power.—By operating storage to the best advantage of the plants as a series, it is estimated that with complete development from 7800 to 7450 minimum or primary horse power (24 hr., 365 days, on wheel shaft) will be available, with secondary power of about 800 hp. This would correspond to from 41.6 to 39.8 mill. kw.-hr. annually, at market, of primary power and from 4.6 to 3.8 mill. kw.-hr. annually of secondary power.

Cost of Development and Power.—The estimated cost of Bancroft reservoir (with high-water level at an elevation of 1260) is \$660,000; the power developments will cost from \$1,535,000 to \$1,661,000 (depending upon the method considered); there has also been allowed \$150,000 for transmission lines. These are preliminary estimates and subject to some uncertainty, as no borings have been made at dam sites, and detailed surveys have not been made as yet for all dam sites.

The cost per kilowatt-hour on the above basis, and considering secondary power at one-half primary, varies from 0.71 to 0.73 ct. There is, therefore, little difference in the comparative feasibility of the methods of development considered.

Value of Water Power Rights.—The cost of about 0.72 ct. per kilowatt-hour includes no allowance for land or property damages or for land and water rights belonging to the Westwood Lumber Company. It is probably conservative to assume that the power is worth 0.8 ct. per kilowatt-hour, or a margin above cost of 0.08 ct. per kilowatt-hour. On this basis 43.5 mill. kw.-hr. annually would yield about \$35,000 which capitalized at 10 per cent is \$350,000, which would represent the value of the entire water privilege, provided all necessary land, property, or rights essential in development are included.

In the above analysis no allowance is made for the value of developments upon Red River in combination with one or more plants on the main Rapid River (or even some other river in North Carolina). Furthermore, the storage at Bancroft would materially benefit existing plants and water privileges on the Rapid River. About 151 ft. of fall is now developed, and the increase in primary power at these plants is estimated at 1500 hp. as a total, or 8 mill. kw.-hr. annually changed from secondary to primary power. At 0.2 ct. per kilowatt-hour this is worth \$16,000 annually. Any adjustment of expense of reservoir on Red River that can be made with parties downstream will increase the value of the Red River privileges.

If \$350,000 is assumed as the value of the Red River water privileges, with all necessary land rights, at 6 per cent this would add an annual charge of \$21,000 and increase the cost of power to 0.77 ct. per kilowatt-hour. This last figure is somewhat less than the cost of steam power in large units at Portland and decidedly less than the cost of power by ordinary steam plants distant from tidewater.

The valuation of \$350,000 is, therefore, reasonable, but it must be considered as only a rough value until more complete information regarding cost of development and probable power output is available.

Land to Be Withheld.—It is evident that the water rights are far more valuable than the land adjacent to Red River, and, therefore, ample reservations should be made for power purposes.

At Bancroft reservoir site, land should be reserved up to an elevation of 1280 (and in a few cases on steep slopes a little higher). For the power sites along the river, land should be reserved to include with some margin all of the various methods of development shown on the plans. Whether strips of land can be reserved for penstock lines or complete blocks, including such lines, must be reserved is a matter not yet determined. The entire river banks should be included to protect water rights.

Concluding.—The complete Red River project is a large one, and it will probably be some time before development is practicable. It is desirable meanwhile to procure more accurate data of the yield of Red River, which is very probably more than we have estimated. We have established gaging stations on Red River at Bancroft and at the Half Way Bridge and these should be continued if possible for several years. The information thus obtained will be of vital importance in planning developments or in a more accurate valuation.

The following pages contain some of the essential details of our investigations and with the accompanying set of plans (sheets 2031.3 to 2031.14, inclusive) form a part of this report.

Respectfully submitted,
E. W. HOLTWOOD & Co.
BY L. R. HOLTWOOD.

OUTLINE OF REPORT

Purpose of investigations.

Surveys.

Results of surveys:

Bancroft reservoir site, areas and capacities.

Amount of fall in Red River.

Gagings of Red River.

Yield of Red River:

Drainage areas.

Rainfall.

River gagings.

Probable available discharge.

Study of proposed developments, summary of results.

Cost of developments, summary of cost.

Cost of power, summary of results.

Effect of lesser reservoir development.

Value of water power rights.

Land to be withheld:

Reservoir site.

Red River power sites.

Concluding.

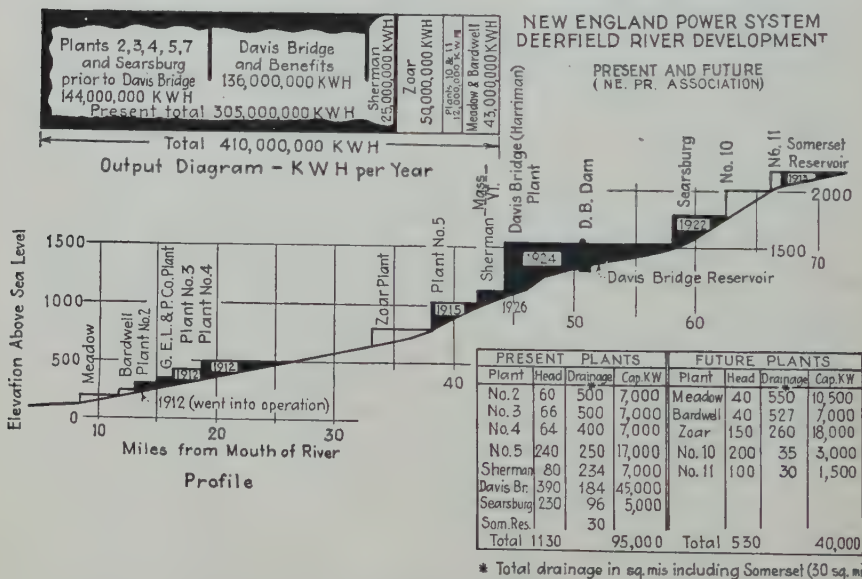
APPENDIX C

PLANT DESCRIPTIONS

DAVIS BRIDGE (Harriman)

I. General.

1. Located on the Deerfield River in town of Whitingham, Vt.
Drainage area, 184 sq. miles.
2. Owned by New England Power Company.
3. Built 1922-1924. Third unit installed 1925.
4. 200-ft. semihydraulic-fill earth dam; 2½ miles of 14- by 13-ft. tunnel in rock, three 9-ft. steel penstocks, 650 ft. in length; short tailrace.



II. Dam.

A. Location and site:

1. Located on Deerfield River in town of Whitingham, Vt., about 5 miles north of Massachusetts-Vermont line. Drainage area = 184 sq. miles, of which 30 sq. miles is controlled by Somerset reservoir (2700 mill. cu. ft. storage), located about 15 miles upstream.
2. Foundation, hardpan with some clay; ledge outcrop at and near surface on left shore and bank; very deep under river and on right bank.
3. Used for power development and storage (about 5 bill. cu. ft.).

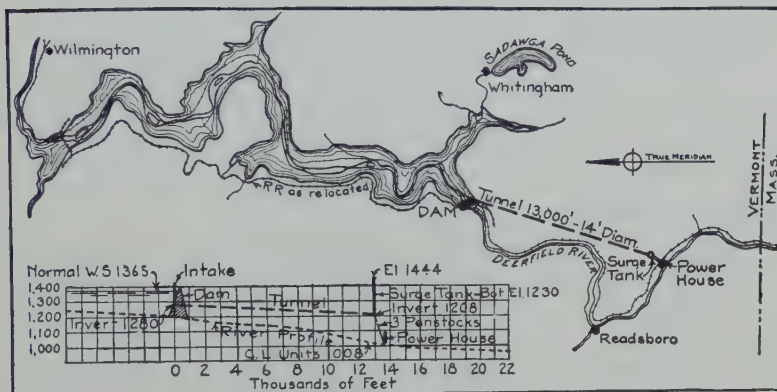


FIG. 285.—Davis Bridge plant, general plan.

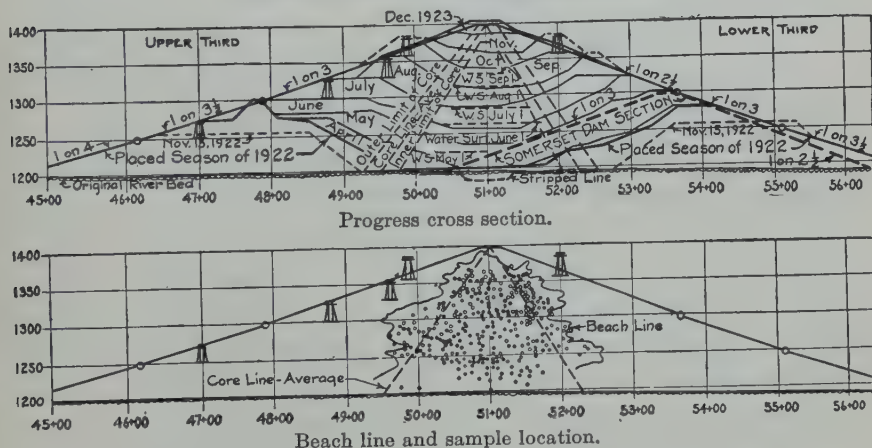


FIG. 286.—Davis Bridge dam, sections—New England Power Company.

B. Main features:

1. Type, semihydraulic-fill earth dam, with tunnel spillway.
2. Dimensions:

a. Section:

Top width 25 ft. at an elevation of 1400.

Upstream slope, at an elevation of 1400 — 1300 = 1 on 3.

1300 — 1250 = 1 on $3\frac{1}{2}$.

1250 — 1200 ± = 1 on 4.

Downstream slope, 1 on $\frac{1}{2}$ steeper than upstream.

- b. Length, 1250 ft. at top.
- c. Elevations:
 - Top, 1400.
 - Spillway, 1386.
 - River bed, 1200 $\pm$.
3. Materials: Sand, gravel, and boulders and some clay—excellent. Central core formed by washing down fill by water jet, effective size, 0.01 mm. $\pm$.

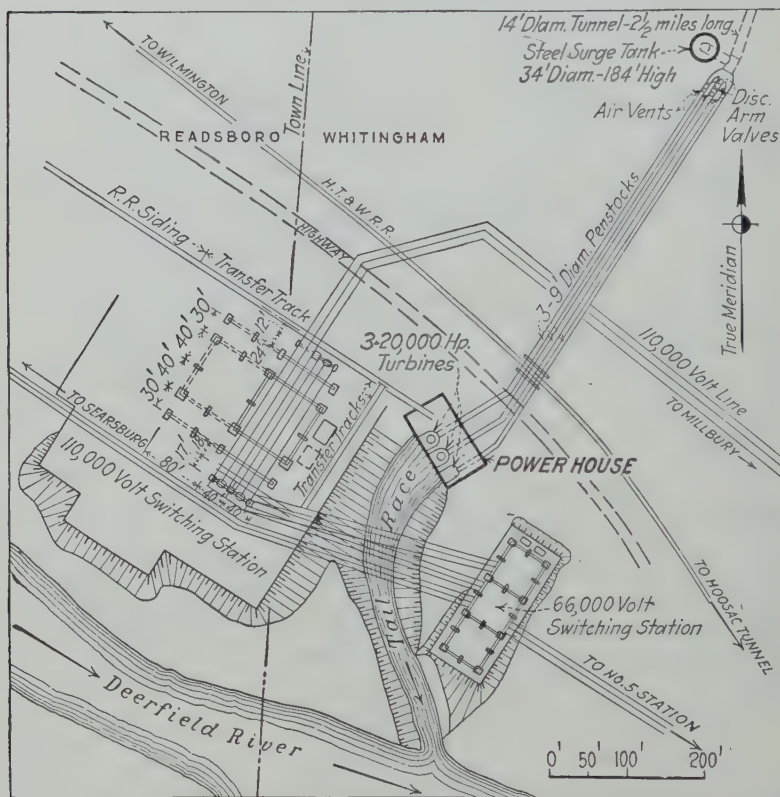


FIG. 287.—Davis Bridge plant, layout near power house—New England Power Company.

4. Spillway: Circular concrete spillway about 300 ft. upstream from center of dam on left bank, with vertical shaft connecting with tunnel through rock under dam to about 300 ft. downstream from lower toe of dam. Spillway, 160 ft. in diameter at an elevation of 1386; 22.5 ft. in diameter at an elevation of 1316; thence vertical drop of about 50 ft.; then 90-deg. bend with 55-ft. radius with center at an elevation of 1215.7; thence sloping to outlet at an elevation of 1200 (invert); total length from an elevation of 1316 to end about 1500 ft. Capacity of spillway about 30,000 sec.-ft. with head of 7 ft. on crest.

C. Accessories:

1. Waste gate: A diversion tunnel, 22 ft. in diameter, 700 ft. long through rock on left bank, to handle water during construction, connecting with spillway tunnel. Concrete plug and 48-in. Dow gate set in lower end of

diversion tunnel when dam was completed in 1924 and used temporarily for river flow while reservoir was filling. This valve left in place and accessible by shaft from gate house.

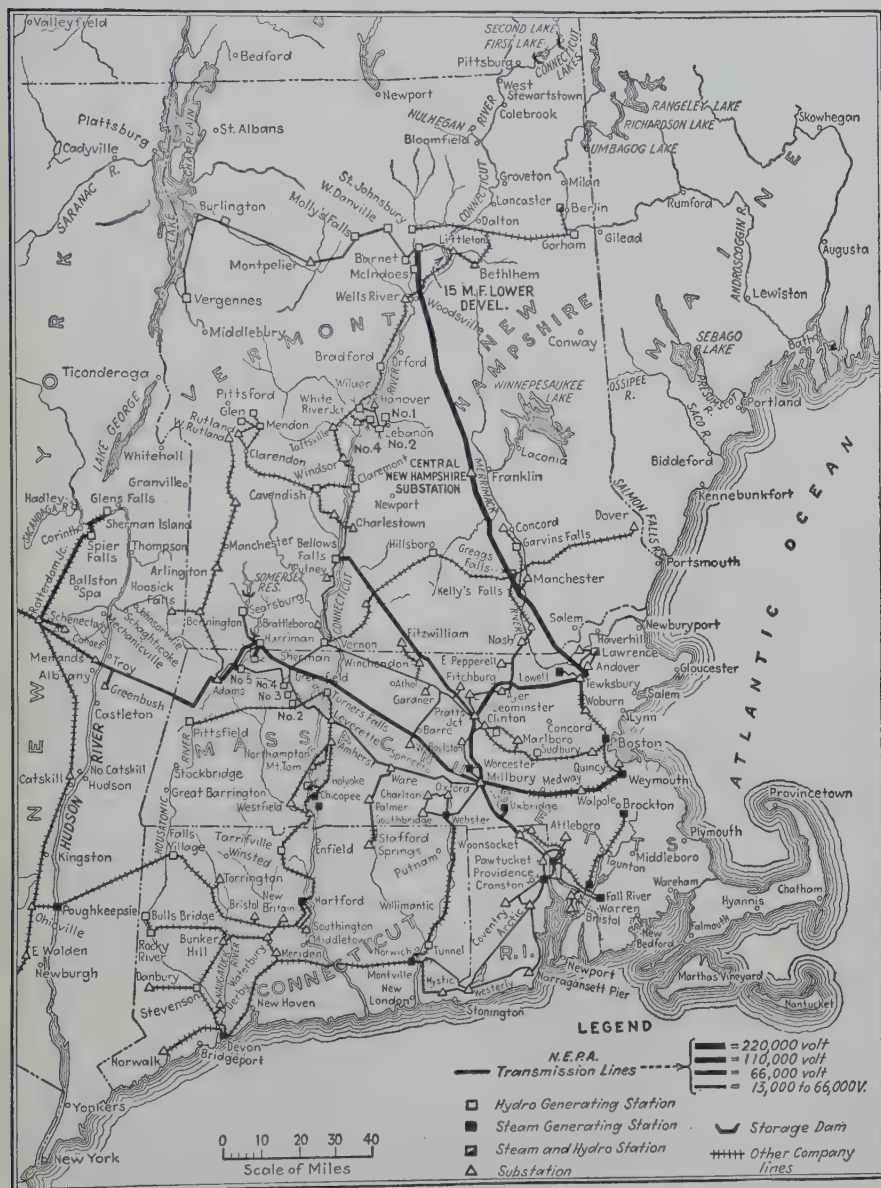


FIG. 288.—New England Power Association, transmission system.

2. Flashboards: Wooden stop logs supported by steel needle beams which can be tripped from a bridge about 10 ft. above spillway level joining the 16 piers and the circular spillway, 160 ft. in diameter. Each pier span

has three needle beams which can be tripped from the bridge by a top latch, releasing the stop logs, which go out. The beams are held loosely

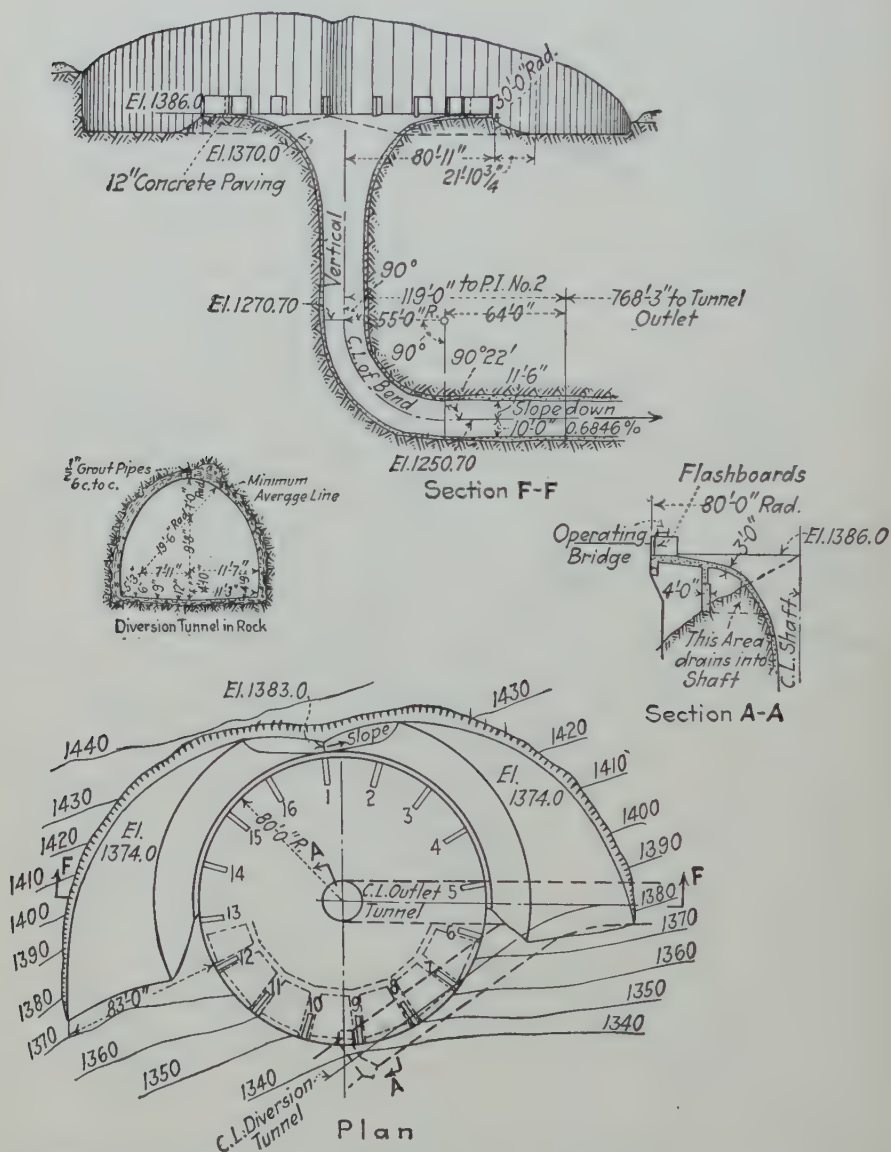


FIG. 289.—Tunnel spillway, Davis Bridge development—New England Power Company.

by chains and can be lifted and replaced. Flashboards carried up to 6 ft. high (see Fig. 227, Chap. IX).

3. No logway.
4. No fishway.

D. Flood capacity: For crest flood of 52,000 sec.-ft. (337 sec.-ft. per square mile) and 24-hr. flow of 24,000 sec.-ft., with spillway maximum capacity of 30,000 sec.-ft. there would be a freeboard of 4 to 5 ft. below an elevation of 1400 (top of earth dam).

E. Storage capacity.

Elevation	Area, acres	Total storage above an elevation of 1300	Remarks
1392		5.0	Top flashboards
1386	2200	4.5	Spillway level
1375		3.5	
1350		1.9	
1325		0.8	
1300		0.0	Power tunnel, approximately

III. Head Works.

A. Gates:

1. Two 8-ft. Dow disk valves, motor operated, in gate well, invert at an elevation of 1280.
2. Area, 100 sq. ft.; velocity (at full capacity, 1700 sec.-ft.) = 17.7 ft. per second.

B. Racks:

1. Rackway 21 ft. wide between gate house walls by 115 ft. high = 2400 sq. ft. Sections supported by 8-in. d.e.h. wrought-iron pipe set horizontally, filled with concrete and spaced 9 ft. on centers. A center pier for middle bearing. Slope 9 on 1.
2. Bars, 3 by $\frac{5}{16}$ in. spaced $2\frac{1}{2}$ in. on centers.
3. Rack platform in gate house.

- C. Gate house: Gate well of concrete, circular, 20 ft. in diameter, bottom at an elevation of 1280, top (floor of gate house) at an elevation of 1390. Rack chamber just upstream from gate well. Superstructure, 24 by 38 ft., concrete walls and hand crane. A 14-ft. square conduit extends about 100 ft. from gate tower to reservoir.

V. Penstock.

A. Location and type:

1. Location on left shore, running from gate house straight across bends in river, tunnel section in rock, hard mica schist.
2. 14 ft. wide by 13 ft. high (161 sq. ft. area) lined with concrete. Total length about $2\frac{1}{2}$ miles to surge tank; thence three individual 9-ft. steel penstocks a distance of about 650 ft. to power house.

B. Main features:

1. Tunnel section, rectangle 14 ft. wide by 6 ft. high with 7-ft. radius crown; section based on convenience for construction to get maximum working width at bottom (see Fig. 140, Chap. VI). Head on tunnel outlet 155 ft. static and 200 ft. surge. Plate-steel lining 14 ft. circular, $\frac{1}{2}$ -in. plate backed with concrete used for final 320 ft., merging into a three-branch manifold connecting with the three 9-ft. pipes. Steel-pipe penstocks, 9 ft. in diameter and $\frac{3}{8}$ - to $2\frac{3}{32}$ -in. plate.
2. Cradles of concrete.

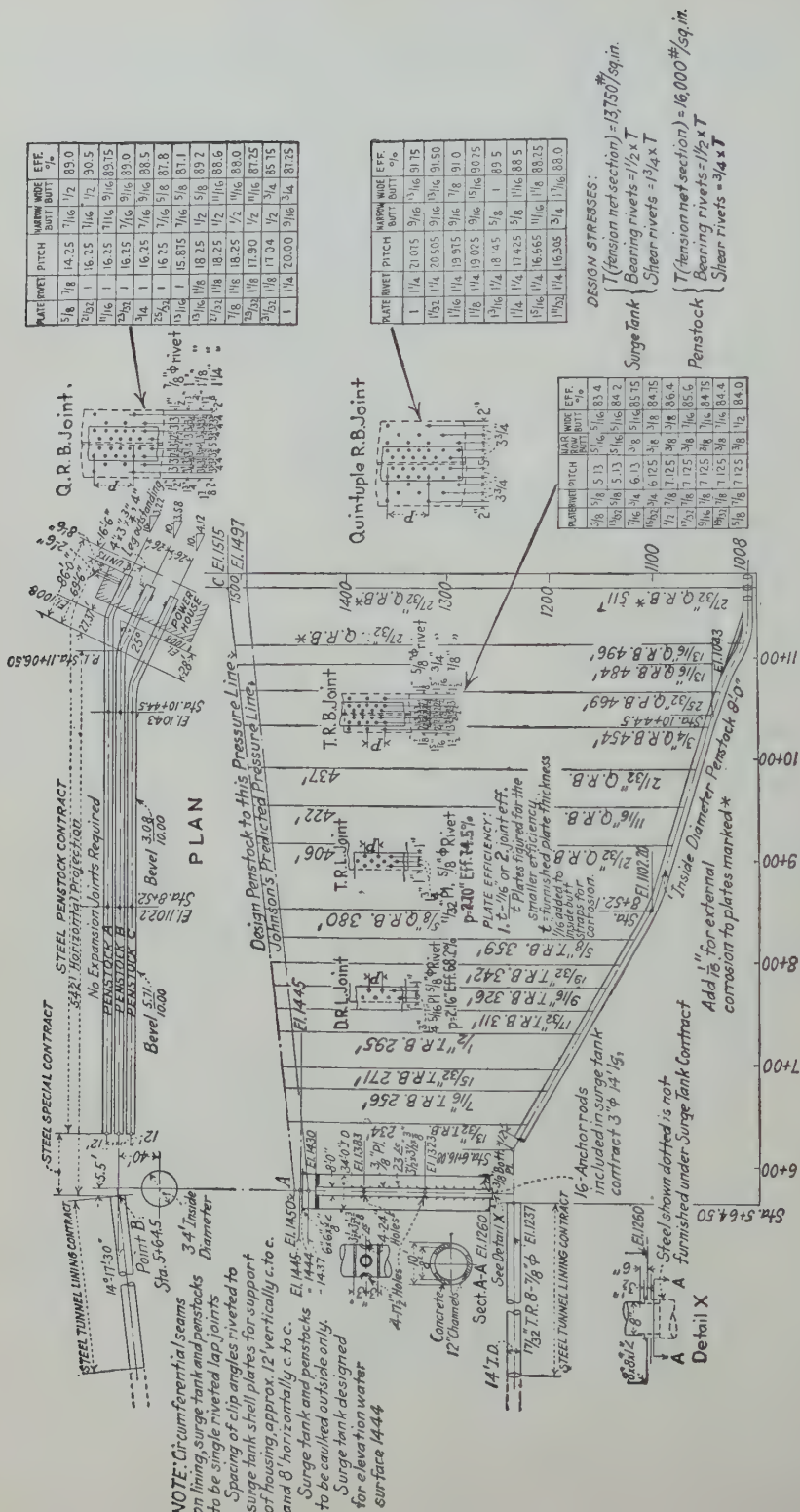


FIG. 290.—Penstock and surge tank layout, Davis Bridge development—New England Power Company.

3. Velocity at full capacity:

In tunnel ≈ 11.0 ft. per second.In 9-ft. penstocks $= 9.3$ ft. per second.

Slope of tunnel 4 per cent for first 1000 ft. (to give desired curve under a brook), thence 0.1 per cent for drainage. Elevation of center at gate

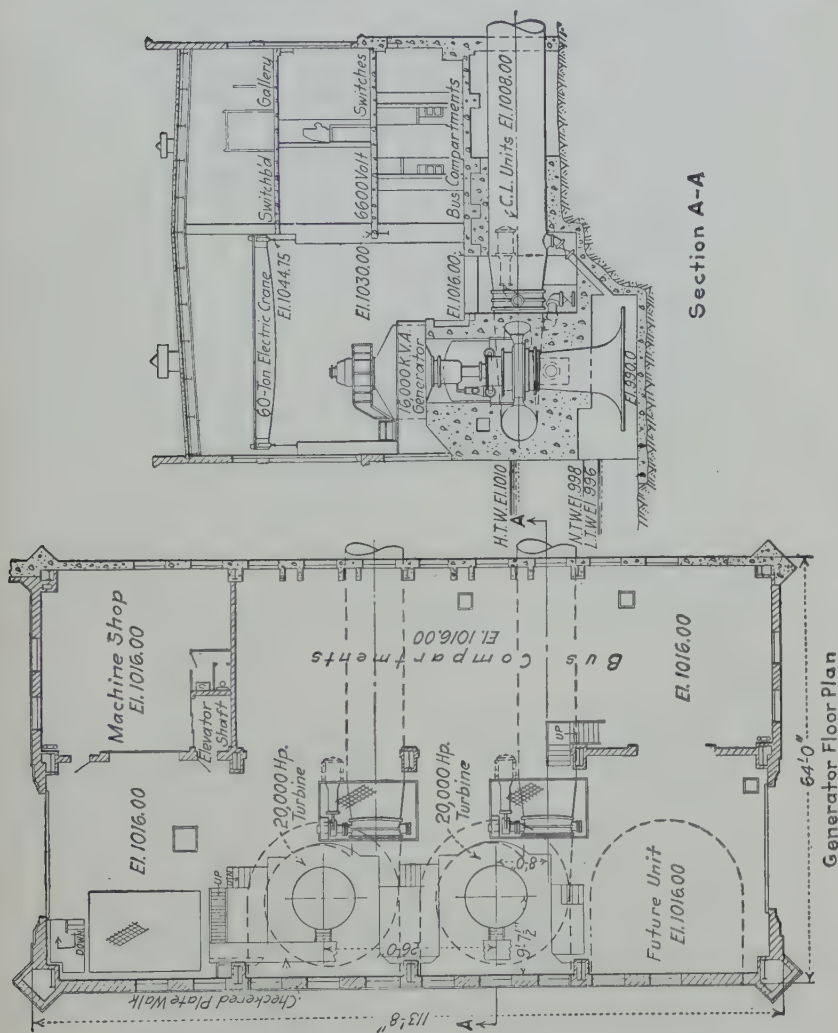


Fig. 291.—Power house, plan and section, Davis Bridge plant—New England Power Company.

house = 1280, at surge tank = 1237. Penstocks at wheel units (center) at an elevation of 1008.

C. Accessories:

1. Surge tank: Johnson differential, near end of tunnel, 10-ft. diameter connection; tank 34 ft. in diameter by 184 ft. high, bottom is 23 ft. above center line of tunnel and 40 ft. to one side; bottom ring of plates $1\frac{9}{32}$ in. thick.

2. Specially designed 24-in. air valves at top of each penstock to vent pipes when head gates are closed.
3. Vent pipe, 4 ft. in diameter, steel, built into head gate house, connecting with tunnel.
4. Gates: 8-ft. Dow disk-arm gates at upper and lower end of each line, motor operated and controlled from power-house switchboard.
5. Other features: Two penstocks first installed, third line installed in 1925.

VI. Power House.

- A. Located about 400 ft. from river, with open tailrace channel. Outdoor transformer and high-tension yards (1) 110 kv. upstream from tailrace, and (2) 66 kv. downstream from tailrace.

B. Substructure:

1. About 64 by 114 ft. on rock foundation, tailrace floor at an elevation of 990; main floor at an elevation of 1016, no basement.
2. Plate-steel scroll flume, inlet 7 ft. in diameter center at an elevation of 1008; hydracone draft tube, bottom at an elevation of 992; tail-pit bottom at an elevation of 990, width 23 ft.
3. Tailrace channel 60 to 30 ft. wide; ordinary water level at an elevation of 998; low-water level at an elevation of 996; high-water level at an elevation of 1010.

C. Superstructure:

1. Main section 32 ft. wide by 110 ft. long; floor at an elevation of 1016, generator base at an elevation of 1022, base-crane rail at an elevation of 1045; underside roof at an elevation of about 1057.
Steel and brick walls, cast stone trimming.
Auxiliary bay 26 ft. wide by 110 ft. long; three floors at elevations of 1016, 1030, and 1044.75 (see plans).
2. First floor, generator room, bus compartments, and machine shop, 28 by 28 ft. elevator.
Second floor, low-tension oil switches and storeroom.
Third floor, switchboard and gallery, office, and lockers.
3. See plans.

D. Equipment:

1. Turbines: Three Allis-Chalmers vertical, single-runner, reaction; 20,000 hp., 350-ft. head, 360 r.p.m. Head varies from about 340 to 390 ft. Efficiency (by test in place) full gate = 87 per cent; full load 90.5 per cent. Governor, Allis-Chalmers.
2. Generators: Three G.E. 16,000 kva., 360 r.p.m., three phase, 60 cycle, 6600 volts, enclosed by steel envelope for ventilation.
3. Exciters, direct connected, 90 kw. each.
4. Switchboard: 15 panels, usual instruments. Oil switches, six sets of compartments for three each; three emergency switches per unit; remotely controlled G.E. type FH 206, 2000 amp., 15,000 volts.
5. Transformers, in outdoor yard; four G.E. 16,000 kva. each 6600/110,000, water cooled.
6. Lightning arresters, outdoor yard. Oxide film.
7. High-tension connections, outdoor yard; galvanized-steel, double-bus type; (1) 66,000-volt equipment downstream side of tailrace; (2) 110,000-volt equipment upstream side.
8. Other features: Crane, 60-ton capacity. Venturi effect of reducer to scroll case calibrated by salt-velocity method and used for record of discharge of wheels.

VII. Costs.

SUMMARY OF COST¹

Item	Cost	Per cent of total
Construction cost:		
Preliminary.....	\$ 208,000	2.0
Dam.....	2,131,000	20.8
Spillway and diversion tunnel.....	1,010,000	9.8
Tunnel and penstock.....	2,662,000	25.8
Power house and tailrace.....	1,368,000	13.2
Miscellaneous, highways, railroad, etc.,.....	1,024,000	9.9
Total.....	\$ 8,403,000	81.5
Administration and engineering.....	572,000	5.6
Interest during construction, etc.....	742,000	7.2
Land and water rights.....	584,000	5.7
Total cost, about.....	\$10,300,000	100.0
Cost per horse power installed (60,000 hp.).....	\$172	

¹ *Elec. World*, Sept. 15, 1923, p. 535, and *Jour. Boston Soc. Civil Eng.*, January, 1925, p. 30.

DAVIS BRIDGE POWER AND RESERVOIR DEVELOPMENT—COST ESTIMATE

Item		Additional general charges	Total for item
Preliminary:			
Investigations:			
Surveys.....	\$ 46,377		
Borings, etc.....	25,698		
Miscellaneous.....	10,000		
	\$ 82,075		
Clearing sites:			
Dam and power house....	\$ 5,809		
Reservoir.....	35,000		
	\$ 40,809		
Construction railroad and			
roads.....	62,000		
	\$ 184,884	\$ 23,000	\$ 208,000
Dam:			
Earth fill.....	\$1,254,000		
Core trench.....	67,500		
Miscellaneous.....	578,500		
	\$1,900,000	231,000	2,131,000
Spillway and diversion tunnel:			
Spillway:			
Excavation.....	\$152,060		
Concrete.....	127,260		
Miscellaneous.....	49,880		
	\$ 329,200		
Diversion tunnel.....	450,000		
Miscellaneous.....	120,800		
	\$ 900,000	110,000	1,010,000
Tunnel and penstock:			
Gate house.....	\$ 120,300		
Tunnel.....	1,750,000		
Surge tank, etc.....	230,000		
Penstock.....	230,000		
Miscellaneous.....	40,650		
	\$ 2,370,950	291,000	2,662,000

DAVIS BRIDGE POWER AND RESERVOIR DEVELOPMENT—COST ESTIMATE
(Continued)

Item	Additional general charges	Total for item
Power house and tailrace:		
Substructure..... \$ 89,640		
Superstructure..... 127,000		
Hydraulic equipment..... 204,000		
Electrical equipment..... 393,000		
Miscellaneous equipment.... 66,380		
Outdoor substation..... 317,900		
	\$1,197,920	
Tailrace..... 22,000		
	\$1,219,920	\$148,000
		\$1,368,000
Miscellaneous:		
Highways..... \$ 112,000		
Railroads..... 735,416		
Operators quarters..... 45,360		
Miscellaneous..... 21,500		
	\$ 914,276	110,000
		1,024,000
Total construction cost \$7,490,000	\$913,000	\$8,403,000

ADDITIONAL GENERAL CHARGES—DETAILS

Item	Amount
Construction camps, etc.....	\$ 71,000
Insurance, watching, etc.....	37,015
Construction plant.....	50,000
Electrical equipment.....	85,000
Contractor's profit.....	670,052
Total.....	\$913,067

VIII. Power Output.

- A. Power used in New England Power Company system.
- B. Station yearly load factor about 0.35; system yearly load factor about 0.42, power and lighting.
- C. Yearly output about 96 mill. kw.-hr. (also 40 mill. kw.-hr. additional at other plants, from storage).
- D. Operating force: Three men, three shifts; three outside men; total yearly cost about \$25,000 (or 0.03 ct. per kilowatt-hour).
- E. Cost of power at switchboard, about 0.6 ct. per kilowatt-hour including benefits of storage at other plants (see reference 2).

IX. Transmission.

- A. Connects with New England Power Company system 76-mile double-circuit 110,000-volt line to Millbury substation; also connecting single-

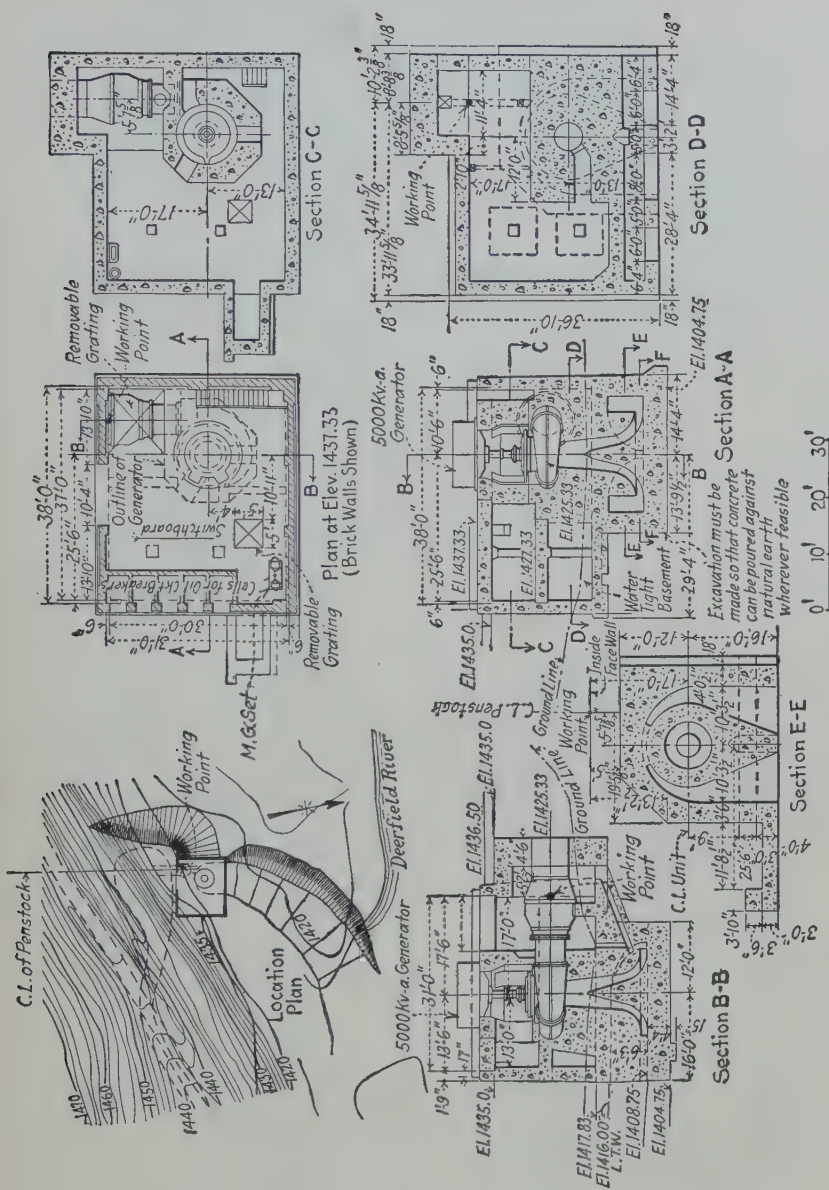


Fig. 293.—Searsburg development, power house substructure—New England Power Company.

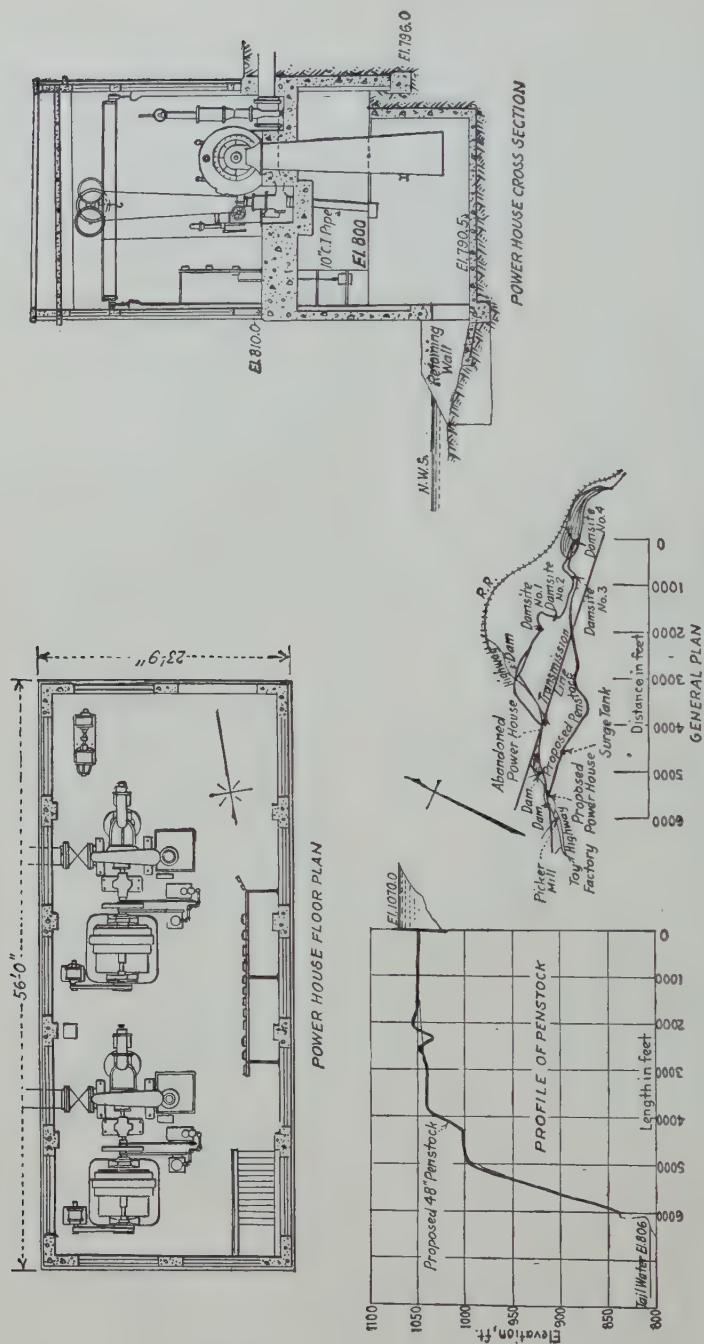


Fig. 294.—Marlboro plant—Keene Gas and Electric Company. General plan and profile and power house layout.
(Courtesy of L. H. Shattuck, Inc.)

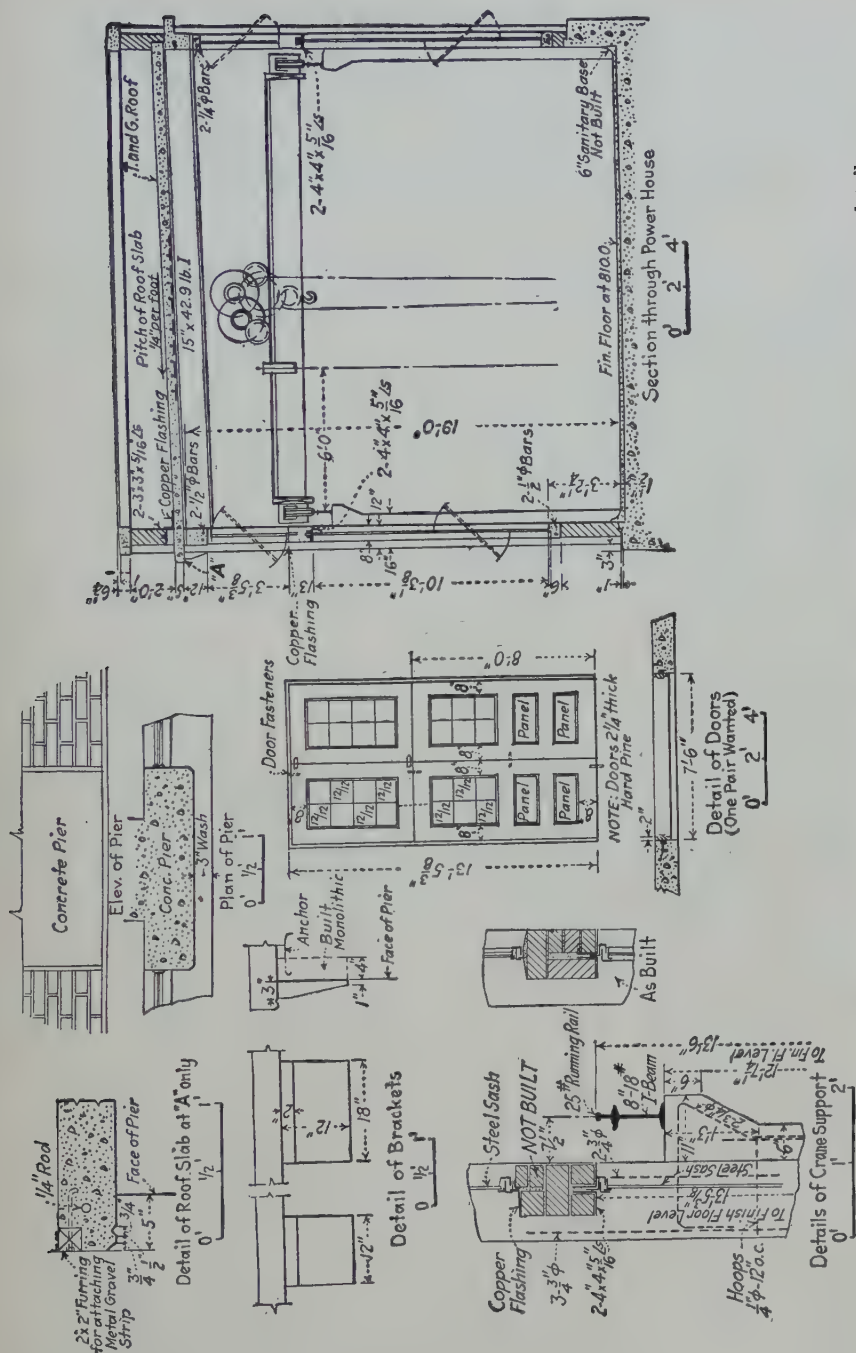


Fig. 295.—Marlboro development—Keene Gas and Electric Company. (Courtesy of L. H. Shattuck, Inc.)

circuit line to Adams, Mass., continuing on to Adirondack Light and Power Company in New York.

- B. Steel towers, 580, 50 to 70 ft. high, to carry six 4/0 seven-strand copper conductors all in one horizontal plane; 60-ft. cross arm, conductor spacing 12 ft.
- C. Outgoing, 64,000 kw., 110,000 volts to Millbury; 32,000 kw., 11,000 volts to Adams; incoming 110,000-volt circuit from plant 6; 66,000-volt line from two adjacent Deerfield plants connects with 110,000-volt yard.

SEARSBURG

- 1. Located on the Deerfield River in the town of Searsburg, near Wilmington, Vt. Drainage area 98 sq. miles.
- 2. Owned by New England Power Company.
- 3. Built 1921-1922.
- 4. 50-ft. semihydraulic-fill earth dam; 18,000 ft. of 8-ft. wood stave, 500 ft. of 6½-ft. steel penstock; short tailrace.
- 5. Capacity 6200 hp. (4000 kw.), one unit, 360 r.p.m., head 230 ft.
- 6. Power used in New England Power Company system.
- 7. Annual output 22 mill. kw.-hr.
- 8. Cost about \$970,000 or \$161 per horse power.
- 9. Equipped for automatic control.
- 10. References: *Eng. News-Record*, Dec. 14, 1922; *Power Plant Eng.*, Apr. 15, 1923. (See Figs. 292 and 293; also Figs. 207 and 218, Chap. IX.)

MARLBORO

- 1. Located on Minnewawa Brook in town of Marlboro, N. H. Drainage area 22 sq. miles.
- 2. Owned by Keene Gas and Electric Company.
- 3. Built 1923.
- 4. 60-ft. concrete arched dam; 5750 ft. of 4-ft. wood-stave penstock.
- 5. Capacity 2500 hp. (2000 kw.), two units, 600 r.p.m., head 270 ft.
- 6. Power used by Keene Gas and Electric Company system.
- 7. Annual output 5 mill. kw.-hr.
- 8. Cost (see details below) \$253,000, or \$100 per horse power.
- 9. Horizontal scroll-case wheel units.
- 10. References:
See Figs. 294 and 295.
- 11. Cost data:

MARLBORO PLANT—COST ESTIMATES

Item	Quantity	Total job cost including equipment and supervision
A. Dam:		
1. Side track.....	165 ft.	\$ 2,642.29
2. Clearing reservoir.....	10 acres	6,053.21
3. Earth excavation.....	680 cu. yd.	3,909.86
4a. Rock excavation.....	590 cu. yd.	8,753.91
4b. Taking care of water.....		2,199.27
5a, b, and c. Concrete.....	2,260 cu. yd. (16,450 sq. ft. forms)	43,149.00
5d. Reinforcing steel.....	21,200 lb.	1,518.80
6a. Sluice gate (30 in.).....		856.66
6b. Penstock gate and screens.....		1,705.23
6c. Dam appurtenances.....		763.24
		\$71,551.47
Preliminary		71.71
Total		\$71,623.18
B. Penstock:		
1. Earth excavation.....	1,576 cu. yd.	\$ 4,191.73
2. Rock excavation.....	1,099 cu. yd.	7,310.97
3. Timber trestle.....	440 ft.	1,787.32
4a. Concrete bases.....	172 cu. yd.	9,660.40
4b. Concrete cradles.....	516 cu. yd.	2,941.50
4c. Concrete forms and bases.....	5,800 ft.	5,220.38
5a. Wood stave, Pacific Tank & Pipe Co.....	5,776 lin. ft.	43,004.12
5b. Hauling and distributing.....	5,776 lin. ft.	4,481.15
6. Venturi meter.....		2,080.54
7. Steel breeches pipe and specials.....		3,921.80
8. Surge tank.....	39,000 lb.	4,220.63
Total.....		\$85,420.54
C. Power house:		
I. Substructure:		
1. Earth excavation including tailrace.....		\$ 8,306.79
2. Rock excavation.....		301.18
3a. Concrete materials.....		3,638.77
3b. Concrete forms.....		2,535.14
3c. Mixing and placing concrete.....		2,512.49
4. Riprap at power house.....		65.30
Total.....		\$17,359.67
II. Superstructure:		
1a. Concrete materials.....		\$ 826.10
1b. Concrete forms.....		4,841.59
1c. Concrete mixing and placing.....		2,859.09
2. Reinforcing.....		688.12

MARLBORO PLANT—COST ESTIMATES. (Continued)

Item	Quantity	Total job cost including equipment and supervision
3. Brickwork.....		821.74
4. Structural steel.....		784.87
5. Doors and windows.....		1,396.08
6. Roofing.....		544.88
7. Crane.....		903.00
8. Grading around power house.....		225.15
		\$13,890.62
Keene Gas and Electric Company charge.....		1,376.57
Total.....		\$15,267.19
III. Hydraulic equipment:		
1. Turbines and auxiliaries.....		\$21,524.24
2. Installation and test.....		1,668.56
Total.....		\$23,192.80
IV. Electrical equipment:		
1. Generator and switchboard.....		\$14,565.82
2. Transmission lines (tie lines).....		2,280.32
Total.....		\$16,846.14
D. Boston and Maine Railroad culvert:		
1a. Concrete materials.....		\$ 136.48
1b. Forms.....		416.54
1c. Mixing and placing concrete.....		1,038.51
2. Reinforcing.....	600 lb.	423.76
3. Taking care of water.....		76.47
Total.....		\$ 2,091.76
E. Highway bridge:		
1. Earth excavation.....	36 cu. yd.	\$ 229.54
2. Rock excavation.....		102.27
3a. Concrete materials.....		64.50
3b. Forms.....		591.42
3c. Mixing and placing concrete.....	75 cu. yd.	591.35
4. Reinforcing.....	1,760 lb.	245.99
5. Riprap.....		80.15
Total.....		\$ 1,905.22
F. Toy-shop Headgate:		
1. Earth excavation.....	21 cu. yd.	\$ 280.76
2. Rock excavation.....		22.40
3a. Concrete materials.....		25.80
3b. Concrete forms.....		332.46
3c. Concrete mixing and placing.....	75 cu. yd.	344.30
4. Taking care of water.....		152.59
5. Headgate.....		278.72
Total.....		\$ 1,437.03

MARLBORO PLANT—COST ESTIMATES. . (Continued)

Item	Quantity	Total job cost including equipment and supervision
<i>G.</i> Substation:		
1. Construction.....		\$ 2,573.13
2. Switching apparatus.....		7,502.49
Total.....		\$10,075.62
<i>H.</i> Riprap on Boston and Maine Railroad:		
1. Materials, freight of.....		\$ 1,716.28
2. Crane rental.....		685.67
3. Boston and Maine Railroad service.....		1,133.11
4. Labor placing.....		691.49
Total.....		\$ 4,226.55

SUMMARY OF COSTS

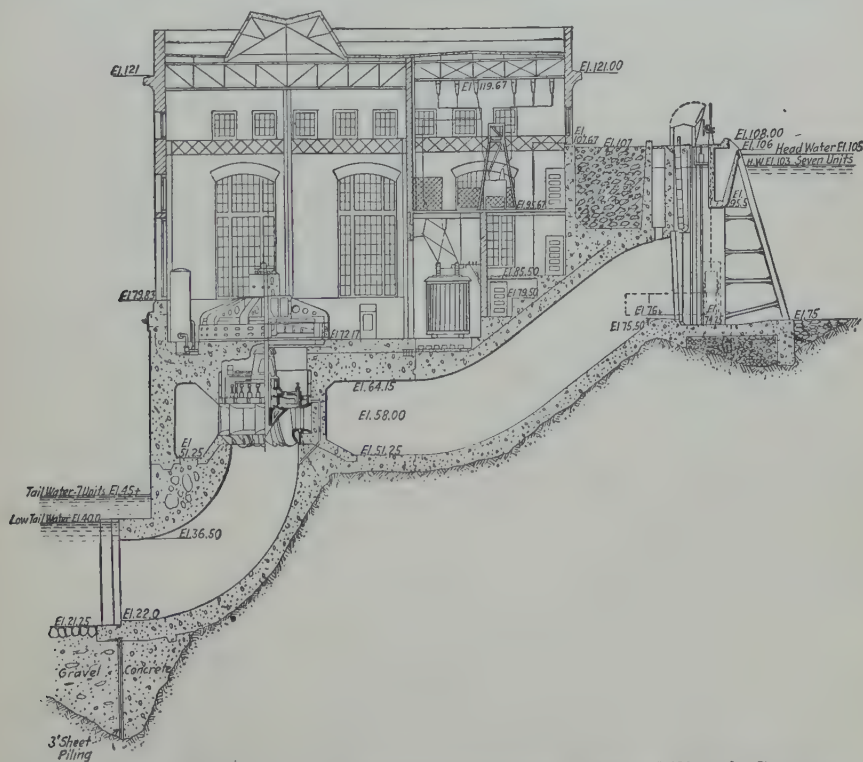
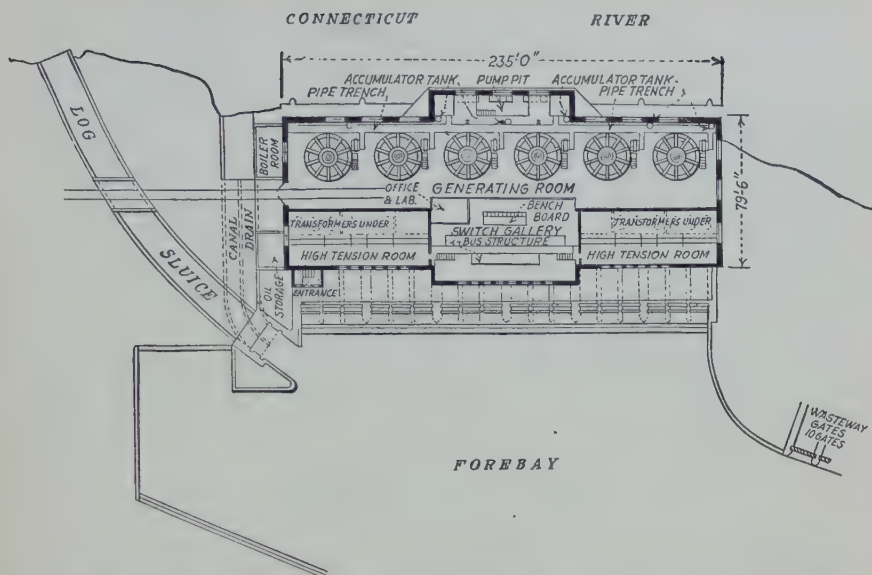
	Total cost	Per cent of total cost
<i>A.</i> Dam.....	\$ 71,623.18	29
<i>B.</i> Penstock.....	85,420.54	34
<i>C.</i> Power plant:		
1. Substructure.....	17,359.37	7
2. Superstructure.....	15,267.19	6
3. Hydraulic equipment.....	23,192.80	9
4. Electrical equipment.....	16,846.14	7
<i>D.</i> Boston and Maine Railroad culvert (dam).....	2,091.76	1
<i>E.</i> Highway bridge (tailrace).....	1,905.22	
<i>F.</i> Toy-shop headgate (tailrace).....	1,437.03	1
<i>G.</i> Substation.....	10,075.62	4
<i>H.</i> Riprap (Boston and Maine Railroad).....	4,226.55	2
	\$249,445.40	100

Rights of way \$ 4,653.19
 Howe reservoir..... 49.65

\$254,148.24

Credit on bridge..... 1,480.00

Total cost..... \$252,668.24
 Plant capacity..... 2500 hp.
 Cost per horse power..... \$101



TURNERS FALLS

Cabot Station

1. Located on Connecticut River at Turners Falls, Mass.
2. Owned by Turners Falls Power and Electric Company.
3. Built, 1915.
4. 30-ft. concrete dam; $1\frac{1}{4}$ -mile canal.
5. Capacity, 70,000 hp. (50,000 kw.), six units, 97 r.p.m., head 60 ft. (formerly 63,500 hp., and 56-ft. head).
6. Power used in Turners Falls Power and Electric Company system.
8. Cost about \$4,800,000 or \$76 per horse power.
9. Spillway flood gates; power-house headgates 10 by 16 ft. of Broome type; mechanical rack valve; large-sized wheels.
10. References: *Eng. News*, July 29, 1915, pp. 202-206; *Gen. Elec. Rev.*, vol. 20, p. 229, 1917. (See Figs. 296 to 298, inclusive; also Figs. 217 and 226, Chap. IX.)

COST DATA FOR ADDITIONAL WHEEL UNIT

Turners Falls Plant, Aug. 5, 1918

Estimated cost of seventh unit at Power Station 2, to be an exact duplicate of No. 6 and to be installed in an extension at the north end of the present power station. (This estimate is based on installing a cofferdam between the power house and the reef, between the fifth and sixth units. The work to be done during low-water period when No. 6 will not be in use. Provision may be made, however, for running No. 6, providing the work is unavoidably delayed until high-water period.)

3873 cu. yd. 1-2-4 stone concrete in foundation at \$15.....	\$ 58,000	
500 yd. concrete connection wall at \$15.....	7,500	
Cofferdam	15,000	
3900 cu. yd. excavation for tailrace at \$1.75.....	6,800	
7000 cu. yd. excavation on site at \$3.....	21,000	
2790 cu. yd. excavation in forebay at \$1.75.....	4,900	
Setting and moving derrick and building trestle.....	5,000	
400 yd. concrete and rock excavation under water at \$35.....	14,000	
Miscellaneous iron work and racks.....	2,500	
Three open-well concrete caissons 5-ft. in diam. at \$1500.....	4,500	
Three Broome gates.....	10,000	
Structural steel.....	8,000	
75 per cent of original cost of building extension.....	17,800	
Removing end wall.....	1,000	
23,000 lb. reinforcing steel at 8 cts.....	1,800	\$177,800

Hydraulic equipment:

Governor and governor tank.....	9,000	
Water wheel (10,000 hp.).....	92,000	
Erection.....	4,000	105,000

Engineering and contingencies, 15 per cent.....	\$282,800
	42,400

Total for building and hydraulic equipment.....	\$325,200
---	-----------

Electrical equipment:

8000-kw. generator, duplicate of sixth unit, including installation.....	\$ 78,000	
Three 3500-kva. transformers, installed and piped, duplicate of present Bank 1.....	41,500	
Switchboard and switching equipment, installed.....	9,500	
Conduit, wire, and cable, installed.....	6,200	
	\$135,200	
Engineering, 5 per cent.	6,760	
Contingencies, 15 per cent.....	20,280	162,440
		\$487,440

$$\frac{\$487,440}{10,000} = \$48.70 \text{ per horse power}$$

CRESCENT

1. Located on Mohawk River about 2 miles above Cohoes, N. Y.
2. Owned by state of New York.
3. Built 1925.
4. 30-ft. solid-concrete dam, concentrated fall.
5. Capacity 8000 hp. (5600 kw.), two units, 400 r.p.m., head 26.5 ft.
6. Power leased to Cohoes Power and Light Corporation; a small amount used for canal operation.
7. Annual output, 44 mill. kw.-hr. (100 per cent load factor).
8. Cost about \$1,900,000, or \$239 per horse power.
9. Plant built as accessory to New York State Barge Canal.
10. References: *Annual Reports* of State Engineer of New York. (See Figs. 299 and 300; also Figs. 167 and 180, Chap. VII.)

BARTLETT'S FERRY

1. Located on Chattahoochee River, 20 miles north of Columbus, Ga.
2. Owned by Columbus Electric and Power Company.
3. Built 1925.
4. 100-ft. concrete dam, 1250-ft. earth sections; 300 ft.-15-ft. steel penstocks (2).
5. Capacity 44,000 hp. (30,000 kw.), two units, 150 r.p.m., head 112 ft.
6. Power used in Columbus Electric and Power Company system.
7. Nineteen 25- by 21-ft. Tainter gates on spillway; four siphon spillways; 15- by 20-ft. steel-concrete headgates with special hoists.
10. References: *Jour. Boston Soc. Civil Eng.*, March, 1926, pp. 93-125. (See Figs. 301 and 302; also Fig. 176, Chap. VII; Fig. 222 and 231, Chap. IX.)

MYSTIC LAKE

1. Located on West Rosebud River, Beartooth National Forest, T 7 S, 16 E, Carbon County, Montana.
2. Owned by Montana Power Company.
3. Built 1924.
4. 1000 ft. of 6- by 7-ft. tunnel; 9000 ft. of 4½-ft. wood-stave penstock; 2730 ft. steel penstock.
5. Capacity 15,000 hp. (10,000 kw.), two units, 300 r.p.m., head 1050 ft.

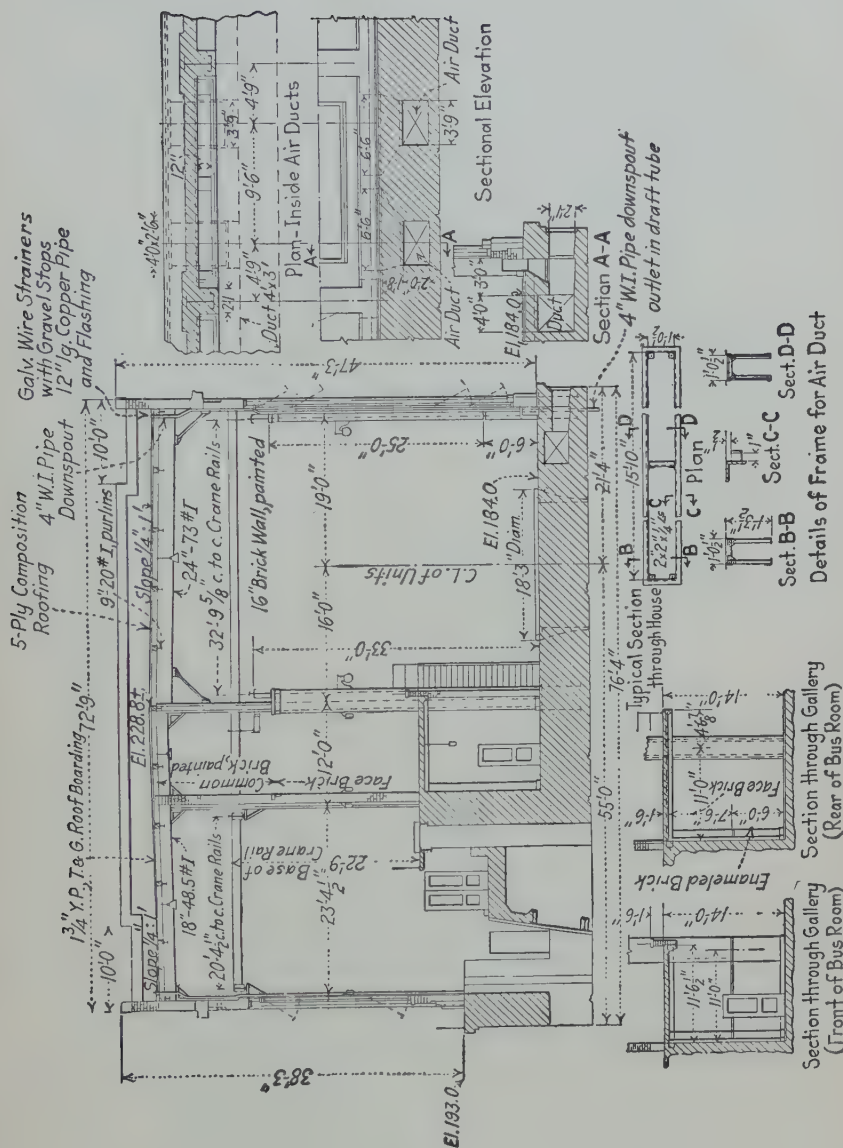


FIG. 300.—Crescent power house superstructure, cross section—State of New York, canal system.

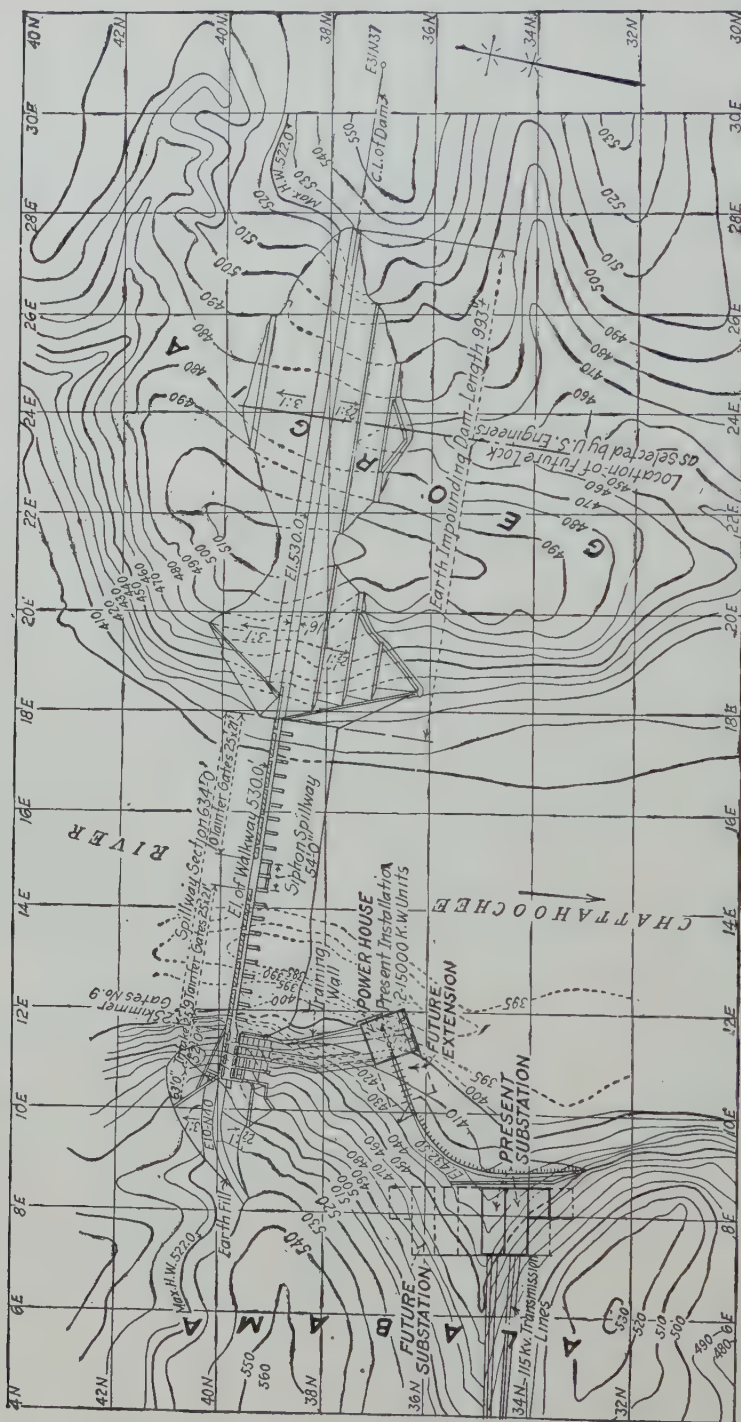


Fig. 301.—Bartlett's Ferry development, general plan. (Courtesy of Stone and Webster Engineering Corporation.)

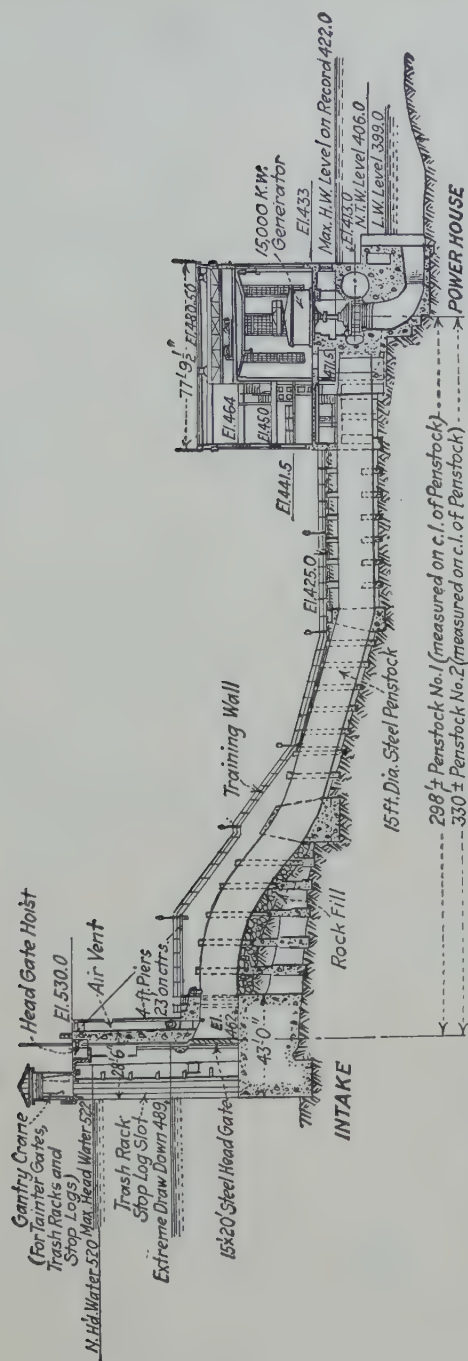


FIG. 302.—Bartlett Ferry plant, profile, penstock and power house. (Courtesy of Stone and Webster Engineering Corporation.)

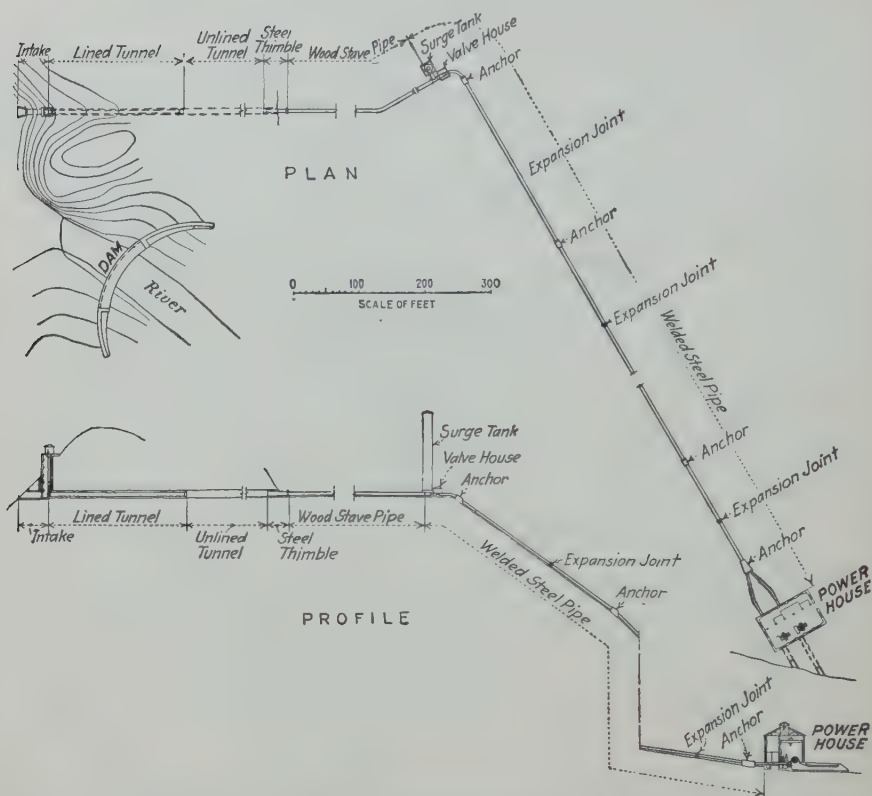


FIG. 303.—Mystic Lake development, Montana Power Company. General plan and profile. (Courtesy of Chas. T. Main, Inc.)

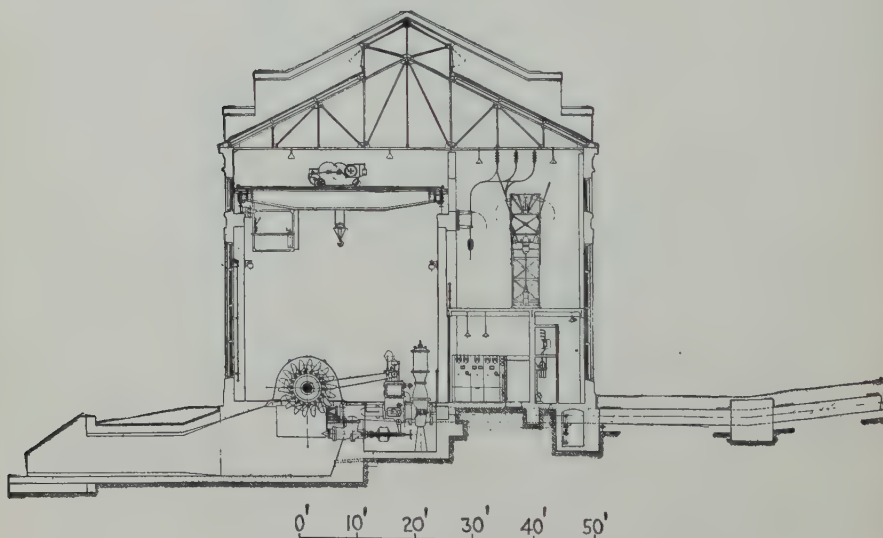


FIG. 304.—Mystic Lake development—power house, cross section.

6. Power used in Montana Power Company system.
8. Cost about \$1,700,000 or \$112 per horse power.
9. Tunnel driven in under lake and outlet made by blasting; single-overhung impulse wheels.
10. References: *Elec. World*, Feb. 7, 1926, pp. 289-290. (See Figs. 303 to 305, inclusive.)

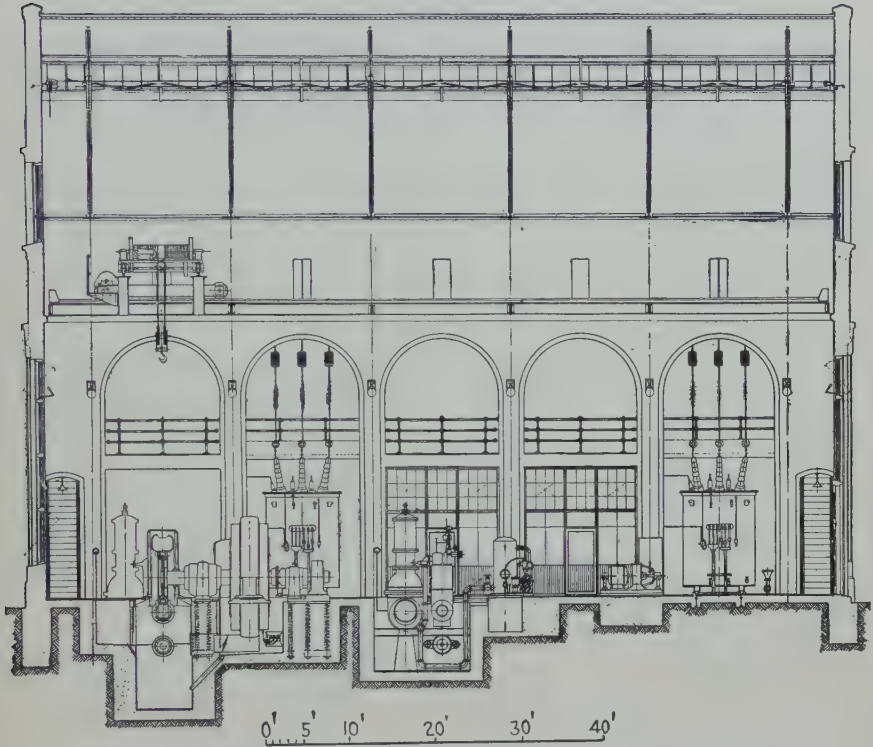


FIG. 305.—Mystic Lake development—power house, longitudinal section.

APPENDIX D

PROBLEMS

The following problems are divided into two groups: (1) a set of miscellaneous problems arranged for the various chapters; (2) a set of problems which include the usual studies, preliminary computations, and designs for a power site and reservoir site on Souhegan River in southern New Hampshire, followed by a review problem (Chaps. I-X, inclusive) of preliminary design and layout for a typical concentrated-fall peak-load plant.

The former are, in general, problems for outside solution by the student, while the latter are adapted for work under some supervision in the class.

PROBLEMS—GROUP I

Chapter II

1. Precipitation—Supplying Missing Data.—The following data of precipitation are taken from the *Journal* of the New England Waterworks Association for June, 1930.

LONG-TIME MEAN MONTHLY PRECIPITATION IN INCHES

Station	February	April	September	December
Nashua, N. H.....	3.54	3.19	3.16	3.41
Brookline, N. H.....	3.43	3.66	3.45	3.49
Winchendon, Mass.....	2.98	3.38	3.92	3.41
Fitchburg, Mass.....	3.29	3.18	3.77	3.52

MONTHLY PRECIPITATION IN INCHES

Station	February, 1917	April, 1917	September, 1918	December, 1918
Nashua, N. H.....	2.58	2.02	7.74	2.81
Brookline, N. H.....	2.87	2.48	7.06	3.26
Winchendon, Mass.....	2.28	1.70	7.99	3.74
Fitchburg, Mass.....	2.31	1.66	7.53	3.01

The location of these stations is shown on Fig. 306.

Assume that the Brookline records are missing for the months February, 1917, April, 1917, September, 1918, and December, 1918, and compute the probable precipitation by the Thiessen method.

a. Using the three adjacent stations with equal weighting.

b. Using the three stations with a weighting inversely proportional to the distance in each case from Brookline.

Compare the computed values thus obtained with the actual precipitation at Brookline.

2. Evaporation from Water Area.—The following data were observed for the month of September, 1907, at the Millinocket, Me., evaporation station (*Water Supply Paper* 279, page 119):

Temperature of air 58.0; temperature of water 59.8°F., relative humidity 79.1 per cent; wind velocity (on raft) 98 miles per day; evaporation 3.72 in.

Compute monthly evaporation, and compare with that observed, by (1) Dalton-Meyer, (2) Bigelow, and (3) Horton formula, assuming $H = 15$ ft.

What will be the approximate "area factor" as used by Horton?

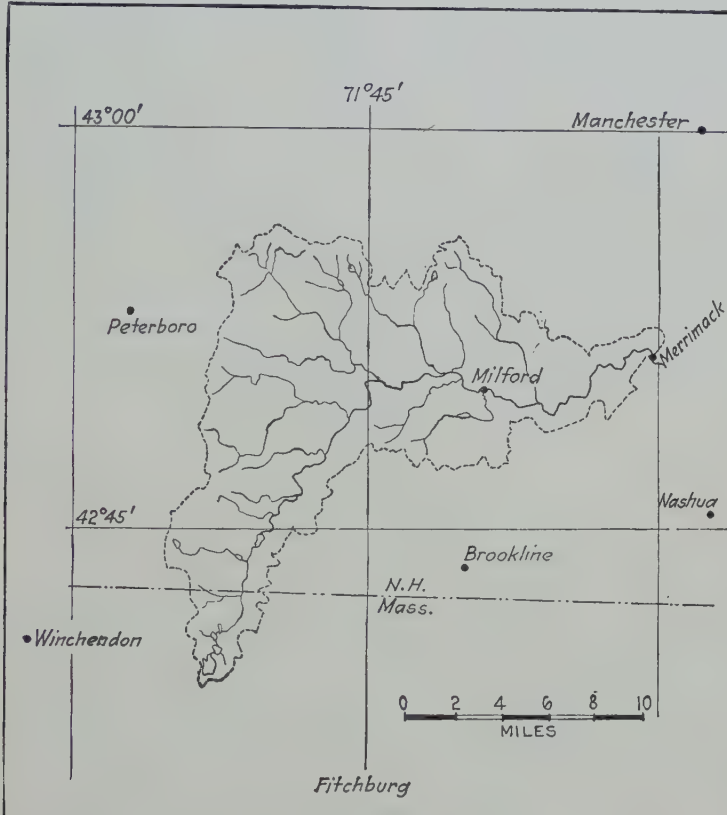


FIG. 306.—Souhegan River—drainage area.

3. Run-off Formulas.—In Table 123 are data of yearly mean precipitation, temperature, run-off, and slope (computed according to Justin's method) for three drainage areas, for the period 1915-1920, inclusive. Compute the yearly run-off from each drainage area by the use of Vermuele's formula and by Justin's formula, and compare the computed and measured run-offs.

4. Drainage-area Characteristics.—Figure 307 shows to scale an area 2 miles square with 100-ft. contours. Determine (1) the average elevation by two methods, (2) the average land slope by two methods, (3) average stream slope, and (4) drainage density, scaling distances approximately.

5. Effect of Water Area on Yield.—The measured yearly run-off of a certain river averaged about 22 in. for a precipitation of about 44 in. for the 10-year period prior to the construction of a large reservoir, during which 10 years the amount of water

TABLE 123.—HYDROLOGICAL CONDITIONS—DEERFIELD, SOUHEGAN, AND NASHUA RIVER BASINS

Item	Deerfield river at Charlemont	Souhegan river at Merrimack	Nashua river at Wachusett reservoir
Yearly precipitation, inches.....	52.83	38.75	44.40
Temperature, degrees Fahrenheit.....	43.8	47.4	47.7
Yearly run-off, inches.....	32.7	23.1	22.8
Drainage area, square miles.....	362	169	119
Slope (Justin).....	0.076	0.031	0.0295

area was about 3 per cent of the total drainage area of 150 sq. miles. Estimate the probable average run-off for this period if the new reservoir, with about 6000 acres of additional water area, had been in use. Assume yearly evaporation from water area at 35 in.

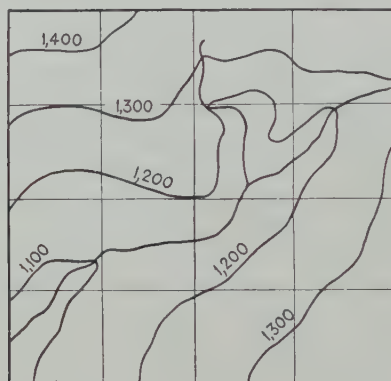


FIG. 307.

6. Flood Flow.—The “average yearly flood” of a river, where the drainage area is 768 sq. miles is 5370 sec.-ft. Using Fuller’s formula, determine the maximum crest floods to be expected on this river for periods of 100, 500, and 1000 years at a point where the drainage area is 133 sq. miles.

Chapter III

1. Flow-duration Curves.—From the data of discharge of Otter Creek, Table 42, page 130, plot a flow-duration curve:

a. With discharge in second-feet plotted against per cent of time and auxiliary scales of second-feet per square mile discharge and wheel-shaft horse power at 80 per cent efficiency for an effective head of 100 ft.

b. With discharge as a per cent of mean flow plotted against per cent of time. Show on the same plot, for comparison, the curve corresponding to $\log Q = 2.40 - 0.011T$ (see Chap. III, page 130). Mean flow is 1.49 sec. ft. per sq. mile.

2. Center of Gravity of Power.—Referring to Fig. 284, Chap. XIII, which shows the present and possible power developments on the Deerfield River, find the center of gravity of power: (A) as developed at present, (B) with all developments completed. Make two sets of solutions, using (a) drainage-area head, (b) plant capacity.

3. Power Output.—Using the center of gravity of power as determined in problem 2 and estimated annual output of 305 mill. kw.-hr. as noted on Fig. 284, determine approximately the average run-off in second-feet per square mile utilized at present on the Deerfield River and compare with the mean run-off at Charlemont as shown by data in problem 3, Chap. II. For utilization factor use an average of the values for the Deerfield River plants in Table 53, Chap. III, weighting these in accordance with plant output. Use overall plant efficiency as 77 per cent.

4. Pondage.—During a low-water week a river has an average daily flow of 1200 cu. ft. per second with a fluctuation during the day requiring a pondage capacity of approximately 25 per cent of the daily discharge. A hydroelectric plant located on the river operates 6 days a week, 24 hr. a day, but supplies power at a varying rate such that the daily load factor is 40 per cent. On Sundays all the flow is ponded for use during the rest of the week.

If the pond area is 1200 acres, and the effective head on the turbines when the pond is full is 60 ft.:

a. What will be the maximum drawdown of the pond during the week if there be sufficient pondage capacity to satisfy all the operating conditions?

b. Assuming no waste or leakage, what will be the weekly output at the switch board in kilowatt-hours? Assume reasonable values for the average efficiencies of turbine and generator.

5. Pondage and Storage.—In a hydroelectric system (see Fig. 308) plants 1 and 2 are operated as a unit but the division of the power output between them is such that the load factor of plant 1 is as high as possible, and plant 2, with large wheel

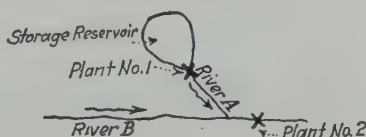


FIG. 308.

capacity, handles system peaks with pondage. The net head on the turbines at plant 1 is 200 ft. and at plant 2 it is 100 ft. The average efficiency of the turbines at each plant is 85 per cent. On a day when only primary flow is available, the uncontrolled flow from river B is 115 cu. ft. per second and there is 260 cu. ft. per second available for discharge through plant 1 from the reservoir. The two plants are so near each other that the time required for discharge from plant 1 to reach the pond above plant 2 is negligible. On the above-mentioned day, the power output from the two plants varies as stated hereunder, and there is no waste of water.

a. What pondage capacity is required at plant 2?

b. What daily load factors are obtained at plants 1 and 2? The system load curve is as shown in Fig. 44, Chap. III, for 50 per cent load factor.

6. Primary Power.—A hydroelectric plant of 6000-hp. capacity with effective head of 65 ft. at full pond has a minimum 24-hr. flow of 300 sec.-ft. and is supplying power under a 50 per cent load factor with energy per cents and hours of use as shown in Table 54, page 179. Pondage is available at the plant of 100 acres, drawn 2 ft.

What horse power at wheel shaft and daily output in kilowatt-hours of primary power is available at this plant with the above conditions?

7. Peak-load Plant.—A power system is operating under a 50 per cent load factor under conditions as shown in Table 54, and on the maximum day has an average daily output of 100,000 kilowatt at switchboard, largely from steam power.

It is proposed to replace peak steam power by hydro-power as far as possible by means of a hydro-station for which the minimum daily 24-hr. available flow averages

1000 sec.-ft. under a head, with allowance for pond drawdown of 50 ft. Pond area is 600 acres.

Based upon data in Table 54, what horse-power capacity of hydro-plant will be required, how many kilowatt-hours of power can be supplied by it on the minimum day, what per cent will this be of the total system daily load, what draft of pond in feet will be required, and under what daily load factor will the hydro-plant operate?

8. Reservoir Effect on Flood Flow.—The proposed reservoir at Gaysville, Vt., on the White River will have a water area at spillway level (elevation 810) of 2400 acres. The spillway is to be 300 ft. long, with values of C as follows:

H , feet	C
0-1	3.3
1-3	3.5
3-5	3.7
5-8	3.9
Above 8	4.0

If a flood should cause inflow to this reservoir with a crest flow of 52,000 sec.-ft. and a distribution of flow during the flood day according to the hydrograph used for Vermont streams (page 182), to what elevation would the reservoir fill (starting with water at spillway level), how many hours would this take, and what would be the maximum spillway discharge, assuming no gate discharge during this time?

Use "mass-curve method" (page 184) in solution.

Chapter IV

1. A double-overhung Pelton-wheel unit is to operate a 30,000-kw. generator, under an effective head at base of nozzle of 1000 ft. About what should be the size of jet, diameter, and revolutions per minute of each wheel? Assume reasonable values for any additional data required.

2. Referring to the record of Holyoke test 2959, page 265, for a 30.2-in. Allis-Chalmers turbine, what are the values of N_u , Q_u , P_u , and N_s when the turbine is operating at best efficiency? What would be the r.p.m., discharge, and power output of a 54-in. wheel of the same series when operating at best efficiency under a head of 70 ft.?

3. Turbines are to be installed in a power house to develop about 10,000 hp. at 200 r.p.m. under a head of 50 ft. What would be the number of units and the horse power and approximate diameter of each?

4. A 45-in. turbine whose characteristics are shown in Fig. 73 (page 219) is to operate under an effective head of 300 ft. What will be the power output, r.p.m., and discharge when operating at full load? What will they be when the turbine is operating at full gate?

5. A turbine of the same series as that whose characteristics are shown in Fig. 80 (page 226) is to develop 8,000 hp. at full gate under an effective head of 55 ft. What will be its diameter, r.p.m., discharge, and efficiency at full gate? If the head lessens 15 ft. under flood conditions, what will be the maximum power output and discharge under these conditions, keeping the same r.p.m.?

6. A 48-in. turbine, homologous with that whose characteristics are shown in Fig. 74 (page 220), is to operate under a head of 70 ft. What should be its r.p.m.? What will be its power output and discharge at full load? What will they be at full gate?

7. A 72-in. turbine, homologous with that whose characteristics are shown in Fig. 75 (page 221), is to operate under a normal head of 22 ft. What should be its r.p.m.? What will be its power output and discharge at full load? What should they be at full gate?

8. Plot on one sheet, for the purpose of comparison, the relation between maximum efficiency and N_s for the turbines whose characteristics are shown in Figs. 73, 74, 75, and 80.

9a. Compute and plot the relation between efficiency and load, as a ratio to full load, for the four wheels of problem 8.

b. From preceding *a* determine and plot range of loads at 80 per cent efficiency against specific speed; also the efficiency-load-range factors.

10. From the record of Holyoke test 2959, page 265, plot (*a*) the power-speed and efficiency-speed curves; (*b*) the characteristic curves.

11. The Safe Harbor plant on Susquehanna River has 220-in. diameter wheel units, rated at 42,500 hp. under a head of 55 ft. at 109 r.p.m., with a discharge of 7700 sec.-ft.

a. Determine the specific speed of these wheels by the method of steps (*i.e.*, reducing values upon the basis of (1) 1-ft. head, (2) 1 hp.). What type of wheels are they?

b. What would be the size of a similar wheel to develop 15,000 hp. under a 15-ft. head, and what would be its speed and discharge?

12. In a certain wheel test with an Alden dynamometer the following data were obtained: $W_1 = 16$ lb. applied 2.24 ft. from center of shaft (initial $W_1 = 2\frac{1}{2}$ lb.); $W_2 = 350$ lb. hung by a lever whose long arm is 4.50 ft. and short arm 0.50 ft. The end of the latter is directly over and connects by a vertical hinged rod with the dynamometer at a distance of 2.32 ft. from its center; r.p.m. = 202, $Q = 61.5$ sec.-ft., $h = 47.5$ ft. Compute wheel efficiency.

Chapter V

1. A masonry dam is to carry a depth of water of 400 ft. Assume that the upstream face has a batter of 1 in 6 and that the dam is the equivalent of a triangle in cross section, with vertex at water surface, and a width of base of 350 ft. Investigate this structure to see whether it is adequate with respect to rules for stability for the cases: (*a*) reservoir empty; (*b*) reservoir full, with no hydrostatic uplift; (*c*) reservoir full, with two-thirds hydrostatic uplift. Arrange all computations systematically. Weight of masonry = 150 lb. per cubic feet.

2. A solid concrete dam with vertical upstream face at a given assumed joint level is 68 ft. thick. The resultant of vertical forces on this joint is 193 tons acting 44.0 ft. from the heel of the dam. The resultant of horizontal forces is 133 tons.

a. What is the maximum vertical compressive stress in tons per square foot and where does it occur? What is the maximum inclined stress if the slope of the downstream face of the dam is 1.0 vertical on 0.8 horizontal?

b. What would these stresses be if the resultant acted 49.0 ft. from the heel of the dam, assuming no tension allowable at the heel?

c. If upward water pressure is assumed effective over one-third the area of the base in preceding *a*, at what distance from the heel of the dam will the resultant force upon the joint level occur? Head on joint at heel of dam equals 50 ft.

d. Under the conditions assumed in problem *c*, preceding, is the dam safe as regards sliding?

3. What should be the cross section (show sketch with dimensions) of a concrete dam, arched in plan with an upstream radius of 150 ft., to retain 100 ft. depth of water? Spillway will be located at an adjacent low spot and need not be provided for in above section.

4. At a certain elevation, a hollow concrete dam, cross section shown in Fig. 309, has a resultant vertical force of 2400 tons per bay, of 18 ft., acting on line *AA*, 47 ft. from heel of dam.

a. Compute the toe and heel maximum vertical stresses in tons per square feet.

b. Determine the thickness of the deck slab at an elevation where the head is 75 ft.

c. How many pounds of steel reinforcement should there be per square foot of slab in *b*? Assume concrete at 650 lb. per square inch, steel at 16,000 lb. per square inch, and weight of steel as 490 lb. per cubic foot.

5. Sketch the cross section required for a timber-framed dam similar to that shown in Fig. 127, Chap. V, and 15 ft. high, noting dimensions and sizes of timbers.

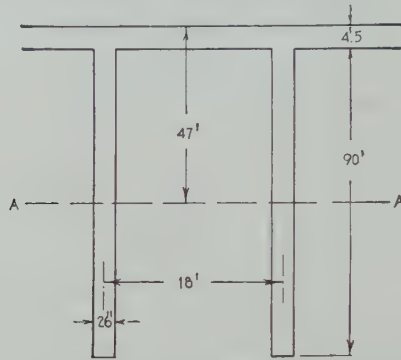


FIG. 309.

6. Given: Data of cross section across a river valley at a dam site as follows:

Station	Ground elevation	Rock elevation
0 + 0	1200	1200
1 + 00	1190	1190
2 + 00	1160	1160
3 + 00	1140	1140
4 + 00	1140	1140
5 + 00	1150	1150
6 + 00	1170	1160
7 + 00	1180	1160
8 + 00	1200	1160

Spillway is to be located at another location and need not be considered.

Two types of dam are to be considered for this site:

Project 1: A concrete masonry dam of bulkhead or non-overfall section with top at elevation 1200 for the full length. Maximum water level is elevation 1190.

Project 2: A concrete masonry dam as in project 1, from station 0 + 0 to 6 + 00, and an earth-fill dam without core wall from station 6 + 00 to 8 + 00, with top at elevation 1200. Earth at site is a compact glacial till with enough gravel and sand to make a good earth-fill dam.

a. Show approximate cross sections with dimensions for concrete dam, earth dam, and retaining wall at station 6 + 00.

b. Determine the approximate cost of each type of dam, stating which will be most economical.

Assume costs per cubic yard as follows:

Project (1):

Concrete masonry.....	\$12.00
Earth excavation.....	1.00
Rock excavation.....	4.00

Project (2):

Earth fill.....	\$ 1.00
Other costs as in project (1)	

Note that preliminary estimates only are contemplated. Give summary of cost by items for each project. Quantities need be estimated only to the nearest 100 cu. yd. Use methods appropriate in accomplishing the purposes of the problem.

Chapter VI

1. A concrete-lined canal for a power development is to be trapezoidal in cross section, side slopes 1 on $2\frac{1}{2}$, and of best hydraulic efficiency. Wheel capacity = 10,000 hp. with efficiency of 85 per cent under a head of 50 ft.; capacity factor = 70 per cent.

a. What should be the dimensions of this canal if its slope is $\frac{1}{1000}$ and C in the Chezy formula is 110?

b. If unlined and in hard earth, about what would be the discharge capacity of the section in *a*, preceding, with the same hydraulic slope?

2. A concrete-lined canal for a power development is to be trapezoidal in cross section with side slopes 1 on 2.5 and of maximum hydraulic efficiency. Wheel capacity is 12,000 hp. with 85 per cent efficiency head 40 ft., and capacity factor 50 per cent. What should be the approximate dimensions of this canal if its slope is $\frac{1}{1000}$ and $n = 0.014$? Use the Manning formula

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

as a basis for solution.

3. A wood-stave penstock line is 12 ft. in diameter and is to carry a head of 60 ft. Staves are $3\frac{1}{2}$ in. in thickness.

a. If $\frac{3}{4}$ -in. round steel bands are used, what should be their spacing?

b. What maximum size of band might be used for this penstock?

4. A steel penstock ($n = 0.018$), 9 ft. in diameter designed to carry 400 sec.-ft., is to be replaced by a concrete penstock ($n = 0.015$) so as to have the same hydraulic slope. What size should the latter be?

5. A canal is to be designed for the following conditions: length 5000 ft., average discharge 1500 cu. ft. per second, total head 40 ft., and the load factor 75 per cent (24-hr. use, 365 days a year).

The cross section is to be trapezoidal with side slopes 1 vertical on 2 horizontal and of best hydraulic section. The water surface is to be 4 ft. below the original ground surface, which is assumed to be horizontal.

Find the most economical cross section for:

a. An unlined canal, with $n = 0.025$, maximum velocity 3 ft. per second.

b. A canal lined with concrete, with $n = 0.012$, maximum velocity 7 ft. per second, concrete lining 6 in. thick extending to an elevation 1 ft. above maximum water level.

The investigation is to be carried out for the following range of conditions:

Excavation, \$0.75 to \$1 per cubic yard.

Concrete in place \$20 per cubic yard.

Power value, $\frac{1}{4}$ to $\frac{1}{2}$ ct. per kilowatt-hour.

Annual fixed charges (interest, depreciation, taxation, etc.) for (a) 8 per cent, for (b) 9 per cent.

For both the unlined and the lined sections give your conclusions as to the following characteristics:

Dimensions of the most economical section.

Mean velocity for average flow, and mean velocity for maximum flow.

Total head lost for average flow and for maximum flow.

Express also the loss of head for each of these conditions as a percentage of the total head.

Which of the two sections is the more economical?

6. What will be the loss of head and power in the canal designed in the previous problem if a sheet of ice 1 ft. thick forms on the water surface?

7. Kelvin's law for economic size of transmission lines requires that the yearly cost of conductor and of value of power lost be equal. This may be stated in the equation

$$Ka = \frac{K^1}{a}$$

where K and K^1 are constants in a given case, and a is the conductor area in circular mils.

a. Prove the above law mathematically, showing that it is consistent with the basic principle that the yearly cost of conductor and value of power lost should be a minimum.

b. Prove in a similar manner that in the case of a steel penstock the yearly value of power lost should be 0.4 of the yearly cost of pipe, for most economic size (see page 356).

8. Design a wood-stave penstock $13\frac{1}{2}$ ft. in diameter to carry a head of 100 ft., determining proper size and spacing of bands and thickness of staves. Wooden cradles with details as shown in Fig. 161, Chap. VI, are to be used, spaced $4\frac{1}{2}$ ft. on centers. Also suitable concrete footings, for a hard, compact soil, assuming ground level 1 ft. below pipe invert and footing to extend to a depth of 3 ft.

Estimate the cost per linear foot of penstock, giving tabular statement of materials required and showing separate cost of one cradle and its footing.

9. Referring to Fig. 303 and plant description, Chap. XIII, showing the profile of the waterway of the Mystic Lake plant:

a. Estimate the loss of head in tunnel, wood-stave penstock, and steel penstock for (1) full-gate wheel discharge and (2) for 0.7 gate, assuming suitable values of friction loss.

b. Construct the hydraulic grade lines, assuming for (1) in (a) low-water level in Mystic Lake (an elevation of 7614) and for (2) an ordinary water level of, say, an elevation of 7650.

Chapter VII

1. A power-house crane with span of 40 ft. is to have a capacity of 60 tons and weighs as follows:

Bridge and trucks.....	40,000 lb.
Trolley.....	8,000 lb.

If the crane load can be carried within 5 ft. of the runway rail, wheel base of bridge truck is 7 ft., and distance apart of runway supports is 12 ft.:

a. Determine analytically the position of the loading which gives maximum moment on the runway girder.

b. Determine proper section for runway girder.

2. Referring to Fig. 293, Chap. XIII, which shows details of power-house substructure for the Searsburg plant:

a. Determine the approximate quantities in cubic yards of concrete and earth excavation in this substructure; also cost of same, assuming concrete at \$20 per cubic yard and earth excavation at \$2 per cubic yard.

b. Determine approximate cost of superstructure on the basis of volume, by reference to data in Table 104, Chap. XII.

c. Compare and check results of *a* and *b* with total cost of power house and equipment of \$41.30 per horse power, as given in Table 104, Chap. XII.

3. Referring to Fig. 300, Chap. XIII, showing details of superstructure of Crescent plant, check sizes of steel members as follows:

- a. Roof girders and purlins, using a suitable loading for the locality.
- b. Crane girders.
- c. Columns in main bay.

4. Referring to Table 75, page 402, as noted below make plots as follows:

(1) Based upon lines 2, 7, and 33, showing N_s/P_u relation, using outlet diameter.

Scales: horizontal..... 1 in. = 0.0005 P_u
vertical..... 1 in. = 20 N^2

(2) Based upon lines 13 and 33, showing relation of unit spacing and outlet diameter.

Scales: horizontal..... 1 in. = 20 ft. unit spacing
vertical..... 1 in. = 2 ft. outlet diameter

(3) Based upon lines 33 and 40 plus 41, showing relation of power-house width and outlet diameter.

Scales: horizontal..... 1 in. = 20 ft. p.h. width
vertical..... 1 in. = 2 ft. outlet diameter

From above diagrams and assuming length of power house equal to number of units times unit spacing, determine approximate length and width of power house for the Lower Fifteen Mile Falls development on Connecticut River, which has four units of 54,000 hp. at 170-ft. head under 138.5 r.p.m., with outlet diameter about 11 ft. Compare with actual power-house dimensions, which are: length 230 ft., width 73 ft.

5. Referring to the draft tube for the Estes-Rainbow development (Fig. 175, page 415) and wheel characteristics as noted on page 410.

(1) Make vertical section through center of draft tube on a scale of $\frac{1}{4}$ in. = 1 ft.

(2) Dividing conical portion of tube into four sections, determine p/w and $V^2/2g$ for each section point, h_f for each section, and total h_f for whole tube.

(3) Make superimposed plots, as follows, of length along draft tube: Horizontal scale 1 in. = 5 ft. against (vertically)

- (a) Area, square feet (1 in. = 25 square feet)
- (b) p/w , feet (1 in. = 5 ft.)
- (c) V , feet per second (1 in. = 5 ft. per second)
- (d) $V^2/2g$, feet (1 in. = 5 ft.)
- (e) Elevation, feet (1 in. = 5 ft.)

(4) Show vertical section of draft tube, scale $\frac{1}{4}$ in. = 1 ft., if the conical portion is modified:

- (a) To give uniformly varying velocity.
- (b) To give uniformly varying velocity head. Determine total h_f for this tube.

Show by tabulation essential computations.

6. A power site where the net available head is 66 ft. is to be developed for a total flow of about 1200 sec.-ft. Determine preliminary values of power, diameter, and speed of vertical wheel units and kilowatts and speed of generator: (a) with a three-unit installation; (b) with two units.

Chapter IX

1. a. Referring to the 10.5-ton gate hoist shown in Fig. 215, find the approximate time required to fully open a gate 10 ft. high if the crank be turned at the rate of 20 r.p.m. Scale and use approximate diameters of gears and pinions, noting that their scale is 1 in. = 4 ft.

b. What force will be required on the cranks to lift the gate if it weighs 8 tons, and what horse power will be exerted if the crank is turned at 20 r.p.m.? Assume each gear-pinion efficiency at 0.90.

2. The gate shown in Fig. 205, Chap. IX, is placed so that the water stands 4 ft. deep over its upper edge when it is closed.

a. Find the maximum fiber stress in the timbers.

b. Assuming a coefficient of friction of 0.45 (wood across the grain on masonry), find the capacity required for the gate hoist.

3. Check sizes of I-beam supports for the racks of the Searsburg development, Fig. 218, Chap. IX, assuming maximum water level at an elevation of 1654 and racks subjected to water pressure only upon upstream side.

4. The log sluice at the development of the Great Northern Paper Company at Millinocket, Me., is described on page 513 and in Fig. 232, Chap. IX. If the water flows 2 ft. deep in the upper rectangular section, what is the discharge? How deep will the water be in the wooden section between stations 4 + 60 and 13 + 45? Use the Manning formula for flow in open channels with suitable value of n .

5. The Tainter gates at the McIndoes Falls plant have a radius of 28 ft. 0 in. to their water face, and the shaft of the gate is 10 in. in diameter with center at elevation 446. Gate sill is at elevation 430, top of gate at elevation 455, and water level, gate closed, at elevation 454. Width of gate 25 ft. 2 in. Total weight of gate is 43,654 lb. acting 22.65 ft. from center of shaft; side sealing strips are 3 in. in diameter. The gate hoist has a gear ratio of 2415/1, and its efficiency is about 50 per cent.

a. Compute the lifting resistance of the gate, if applied 28 ft. from center of shaft.

b. What horse power of motor at 900 r.p.m. will be required for lifting the gate?

c. How long will it take to raise the gate from closed position to elevation 455?

Chapter X

1. A 2000-hp. wheel running at 360 r.p.m. and set in an open flume has a WR^2 of generator and wheel of 84,000. If complete gate closure occurs in 4 sec., what will be the percentage speed variation?

2. A certain plant has conditions as follows: wheel horse power 2000; $h = 100$ ft.; penstock 10 ft. in diameter and 700 ft. long of $\frac{3}{8}$ -in. steel pipe; no surge tank or relief valve.

What will be the rise in feet of pressure head due to complete closure of wheel gate by the governor in 4 sec.?

What will be the drop of pressure head due to acceptance of complete load in the same time?

3. If a simple surge tank 25 ft. in diameter is to be installed near the wheel in problem 2, what should be its top elevation? What minimum elevation would the water level reach in the surge tank after complete load acceptance?

What would be the approximate diameter of a differential surge tank for the same conditions?

Chapter XI

1. Given the following conditions for a wood-pole H-frame line: three-phase three-wire line; 40 miles long; load 5000 kw.; power factor 0.8; delivered voltage = E_r = 44,000; frequency 60 cycles; stranded bare hard-drawn copper.

Required: Size of conductor for not more than 5 per cent line losses, allowable line drop; power factor at sending end; conductor spacing, voltage regulation and sketch of conductor arrangement.

2. On page 567 is a statement regarding the occurrence of corona upon a 1/0 conductor at sea level and at 9000 ft. elevation.

Check this statement by the use of Eq. (12), page 568.

3. The 220-kv. transmission line from the Fifteen Mile Falls plant at Munroe, N. H., to Tewksbury, Mass., is a two-circuit line of A.C.S.R. type with conductors 1.09 in. in diameter.

Compute E_0 , E_v , and P for corona effect by the formulas on page 568 for both fair weather and storm conditions, assuming $b = 29$ in. (fair weather); 28 in. (storm); $D = 23.5$ ft.; $t = 32^\circ\text{F}$.; $m = 0.85$; $M_v = 0.82$.

4. Construct on standard cross-section paper Thomas sag and length curves based upon data in Table 94, as follows:

Values of l : range 1.00 to 1.02; scale at bottom of sheet, 1 in. = 0.002 (length of $8\frac{1}{2}$ - by 11-in. sheet)

Values of λ : range 0.0 to 0.09; scale at top of sheet, 1 in. = 0.01

Values of K : range 0.0 to 7.0; scale at left of sheet, 1 in. = 1.0

5. A transmission line of 3/0 conductors, of stranded hard-drawn copper has spans of 800 ft. between supports at the same elevation and is designed for class B loading.

a. Determine sag, length of conductor, maximum pull, and maximum horizontal stress for conditions of maximum load.

b. Determine same at 70°F . with bare wire.

c. Determine same at 70°F . with class A wind loading. Use Thomas sag curves from problem 4 in the solution.

6. Determine sag and length of conductor by parabolic formulas, in problem 5b.

7. If the supports in the transmission line in problem 4b are at a difference in elevation of 70 ft., determine the maximum sag and its location, using in the solution Fig. 267, p. 607.

8. Show that for a wooden pole set in the ground, as used in transmission lines, the weakest section in cross-bending is where the diameter is one and a half times the diameter where the load is applied.

Chapter XII

1. Assuming that the cost of equipment, etc., required to add to the capacity of a water power plant is \$60 per horse power and coal is worth \$5 per ton of 2000 lb., about how many months of the year will it pay to use the water power, based upon coal saving?

2. A reservoir development of 1.1 bill. cu. ft. capacity will supply water for a total developed head of 400 ft. with wheel capacity of 7000 hp. and will cost \$1,200,000. If coal is worth \$6 per ton of 2000 lb., primary power 1 ct. per kilowatt-hour, and secondary power 0.33 ct. per kilowatt-hour, determine the advisability of use of this storage (a) based on coal saving and (b) based on increase in primary power. The flow-duration curve for the average year at the center of gravity of power is as follows, prior to use of storage:

Per cent of time.....	0	10	20	30	40	50	60	70	80	90	100
Flow, second-feet.....	400	260	180	142	120	100	78	60	50	30	

Assume for the average year that two-thirds of the storage capacity will be effectively used. Use suitable values for any further data required.

3. A water power site where the net available head is 85 ft. has a drainage area of 250 sq. miles. The available unit flow is similar to that of Otter Creek at Middlebury (Table 42, Chap. III). The estimated cost of development on the basis of flow available one-third of the time is \$210,000 not including transmission.

Determine the value of this water privilege by the net income method if the sale price of primary power is 1.0 ct. per kilowatt-hour and secondary power 0.3 ct. per kilowatt-hour at switchboard with a 60 per cent load factor. Assume suitable values for other data required.

4. A hydro-plant of 30,000-kw. capacity can be developed for \$250 per kilowatt (fixed charges 10 per cent) and can deliver 90 mill. kw.-hr. yearly of power to replace steam power, costing \$110 per kilowatt of capacity (fixed charges 14 per cent) and with fixed operating costs of \$6 per kilowatt-year and increment costs of 0.20 ct. per kilowatt-hour. Ample pondage is available so that all hydro-power is firm. An additional 10,000 kw. of hydro-capacity will be firm in 3 years, will have an increment cost of \$60 per kilowatt and would add about 10 mill. kw.-hr. yearly. Yearly cost of transmission for hydro-power is \$3 per kilowatt of capacity.

Determine the relative economy of hydro and steam power in each case.

5. A description of the Rocky River pumped-storage plant is given on page 679. Assume the following additional conditions:

(1) The natural flow of Rocky and Housatonic rivers in second-feet per square mile, for the average year and dry year, by months in order of dryness, is as follows:

Average year:

0.35, 0.40, 0.45, 0.50, 0.60, 0.80, 1.00, 1.25, 1.70, 2.50, 3.20, 4.50

Dry year:

0.20, 0.25, 0.25, 0.30, 0.35, 0.40, 0.50, 0.70, 1.20, 2.25, 4.20, 6.50

(2) A flow of 1800 sec.-ft. must be maintained in Housatonic River when pumping to the reservoir.

(3) Overall pump and motor efficiency is 85 per cent.

(4) Cost of power for pumping to be taken at 0.1 ct. per kilowatt-hour. Primary power from the reservoir will replace peak steam power, with coal at \$4 per ton.

(5) The yearly load duration curve of the power system, in its upper portion, is as follows:

Load in Thousand Kilowatts	Duration, Per Cent of Time
150	0
130	2.5
123	5
118	10
116	15
110	20
96	30
77	40
64	50

Make a study of this plant, and determine approximately what amount can be economically expended for its construction, including cost of land and flowage.

Assume and state suitable values for any additional data required.

PROBLEMS—GROUP 2

I. PRECIPITATION—SOUHEGAN RIVER DRAINAGE AREA

Data: Map of drainage area, reduced from the U. S. Geological Survey topographic sheets (see Fig. 306).

Annual precipitation at stations near drainage area from "Rainfall in New England," by Goodnough in the *Journal* of New England Water Works Association, September, 1915, page 237, and September, 1921, page 228, in Table 124.

TABLE 124.—ANNUAL PRECIPITATION—INCHES

Year	Nashua, elevation 125	Brookline, N. H., elevation 250	Winchendon, elevation 975	Peterboro, elevation 744	Manchester, elevation 200	Fitchburg, elevation 625
1890	53.02				44.56	51.85
1891	39.17				35.25	45.58
1892	36.40			39.18	34.89	41.63
1893	42.57	47.01		41.99	37.93	48.82
1894	29.50	32.11	31.31	36.44	26.67	33.13
1895	40.92	45.23	40.14	42.44	40.15	45.25
1896	37.83	40.59	37.27	37.39	36.25	39.39
1897	45.83	52.23	51.14	49.18	46.64	51.29
1898	51.14	50.40	50.64	51.47	46.05	55.71
1899	37.12	40.89	39.06	38.61	35.26	37.73
1900	46.59	50.79	48.44	50.22	46.91	50.22
1901	43.75	49.52	45.97	50.92	47.10	54.17
1902	49.92	47.81	46.05	49.23	48.32	47.64
1903	41.80	44.51	44.13		42.01	43.88
1904	37.62	40.66	40.40		36.14	40.01
1905	35.05	43.26	41.88		38.61	43.85
1906	42.21	42.60	37.65		43.34	45.58
1907	40.61	40.88	41.11		42.60	44.52
1908	35.49	35.64	30.72		34.07	36.19
1909	35.88	38.44	38.71		35.66	37.60
1910	31.95	32.36	33.25		31.63	35.29
1911	37.61	39.52	38.30		35.66	37.13
1912	36.94	38.66	37.26		38.92	37.55
1913	37.57	40.92	37.01		34.57	38.27
1914	34.01	33.18	35.12		31.21	28.41
1915	35.79	42.33	44.02		40.52	40.46
1916	39.96	45.31	45.93		43.32	37.54
1917	34.50	40.27	43.14		36.71	30.39
1918	33.46	34.05	40.12		33.74	36.95
1919	31.57		44.83		58.61	41.34
1920	49.03		54.79		52.95	50.10
Mean	39.51	41.89	41.42	44.28	(1875-1920) = 38.90	(1871- 1920) = 41.31

FITCHBURG—ADDITIONAL DATA

1870		1875	41.82	1880	32.04	1885	37.60
1871	33.87	1876	38.05	1881	33.84	1886	47.03
1872	40.09	1877	36.05	1882	33.31	1887	55.38
1873	38.79	1878	47.03	1883	27.45	1888	59.60
1874	34.99	1879	38.88	1884	36.27	1889	45.79

Areas of quadrilaterals 15 min. in extent from S. S. Gannett, "Geographical Tables and Formulas," U. S. Geological Survey, *Bull.* 650.

Middle Latitude	Area, Square Miles
42° 37' 30".....	219.91
42° 52' 30".....	219.04

Problem.

1. Determine the scale of the map accurately from the area of a quadrilateral. Graphically, as in Fig. 18, divide the drainage area into the areas governed by each rainfall station shown on the map, and determine the area in square miles for each station, the total drainage area, and the percentage weight for each station.

2. Compute the mean annual precipitation for each station for the years 1894-1902 and 1894-1918. Using these results compute the ratio of the 1894-1902 mean to the 1894-1918 mean for each station, and compute also the mean of these ratios. Then compute the probable 1894-1918 mean for Peterboro, using the above mean ratio.

3. Using the 1894-1918 mean as determined in (2) (including that for Peterboro) and the areas in (1), compute the probable 1894-1918 mean annual precipitation for the entire drainage area.

4. Adjust the results of (3) to give the long-time mean annual precipitation, using Fitchburg as an index station.

5. *a.* Plot frequency curves of annual precipitation at Fitchburg for 1875-1920, inclusive. Note mean and normal annual precipitation in inches and per cent of the mean.

Abscissas: per cent of time..... 1 in. = 20 per cent

Ordinates: per cent of mean..... 1 in. = 20 per cent

b. Plot frequency curve with abscissa on a probability scale with length of about 5 in., ordinates as before.

6. Based on results of (4), and the maximum and minimum precipitation at Boston (page 51) for its 100-year record, determine the maximum and minimum yearly precipitation to be expected on the Souhegan River drainage area.

II. EVAPORATION FROM WATER AREA

Data: Monthly air temperature, relative humidity, and wind velocity for the period 1907-1913, inclusive, at Amherst as appended. Also refer to Table 29, Chap. II.

Problem.

1. Compute and tabulate evaporation from water area for the mean of each calendar month and the year for the period 1907-1913, inclusive; also the mean air temperature, relative humidity, and wind velocity by months and years for this period. Use Dalton-Meyer formula assuming temperature of water surface the same as that of air.

2. Make a plot of mean monthly computed evaporation in inches using months as abscissas, scale 1 in. = 2 months, and evaporation as ordinates, scale 1 in. = 1 in. evaporation.

3. Plot monthly mean temperature on same sheet using scale of ordinates 1 in. = 10°F.

4. Show on the above plot for comparison a curve of monthly evaporation based on (a) Fitzgerald's experiments at Chestnut Hill and (b) monthly evaporation for Rochester, N. Y.

TABLE 125.—OBSERVATIONS AT AMHERST, MASS.

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year
Mean hourly temperature, degrees Fahrenheit													
1907	22.4	16.5	35.2	41.5	51.8	63.9	70.0	66.1	61.3	45.6	37.6	30.5	45.2
1908	25.7	20.5	34.7	45.1	59.2	67.6	72.5	66.6	62.9	51.3	38.0	27.1	47.6
1909	25.7	28.1	32.4	44.4	55.5	66.4	68.7	66.5	60.5	47.7	41.3	24.7	46.8
1910	25.5	23.9	39.1	50.6	56.1	63.8	72.1	67.1	61.1	51.7	36.4	21.7	47.4
1911	27.4	23.3	31.5	43.7	61.9	64.5	73.7	67.8	60.2	48.5	36.7	32.7	47.6
1912	14.6	20.7	30.5	45.2	58.1	65.0	71.6	66.4	61.2	52.4	40.0	32.9	46.5
1913	33.6	22.7	37.9	47.6	55.6	66.4	71.4	69.5	59.7	54.7	51.6	31.3	49.3
Mean.....	25.0	22.2	34.5	45.4	56.9	65.4	71.4	67.1	61.0	50.3	38.8	28.7	47.2
Mean relative humidity, per cent													
1907	76.1	80.2	73.4	74.1	75.3	76.9	76.4	74.9	83.0	77.7	85.9	80.4	77.9
1908	73.8	84.8	77.9	64.3	74.8	66.2	76.6	79.0	79.1	79.0	79.6	75.1	75.8
1909	78.5	78.9	81.3	76.3	71.2	73.6	71.8	78.1	83.1	76.1	77.2	75.2	76.8
1910	80.9	81.3	72.8	69.1	71.6	75.4	70.3	76.3	82.7	75.0	78.7	78.4	76.0
1911	78.8	77.6	73.2	65.2	72.2	74.5	70.7	77.5	82.4	79.0	73.5	77.3	75.2
1912	81.3	78.4	80.3	77.2	78.5	70.0	71.5	78.3	84.0	74.9	77.2	75.4	77.3
1913	79.0	74.5	79.3	74.4	73.0	68.4	70.5	74.2	80.2	79.2	74.0	81.2	75.7
Mean.....	78.3	79.4	76.9	71.5	73.8	72.1	72.5	76.9	82.1	77.3	78.0	77.6	76.4
Average wind velocity, miles per hour													
1907	6.8	7.2	7.8	9.7	8.2	5.8	5.6	5.4	4.9	7.0	6.5	7.2	6.8
1908	10.7	7.6	7.9	11.3	8.0	6.3	5.2	5.2	5.1	5.0	7.5	7.4	7.3
1909	8.2	7.6	9.6	9.1	7.3	5.8	7.0	4.8	5.5	6.0	7.9	8.0	7.3
1910	7.9	8.0	7.6	7.6	7.2	5.0	5.2	5.8	4.6	7.5	7.1	7.4	6.7
1911	8.3	7.6	10.2	7.8	6.7	4.9	5.3	4.1	5.2	4.7	8.1	6.7	6.6
1912	6.7	6.5	7.2	8.3	7.3	6.2	5.5	5.0	4.1	5.4	6.9	7.3	6.4
1913	7.3	7.1	8.8	7.7	5.0	5.1	5.9	4.7	4.4	6.4	6.8	6.0	6.3
Mean.....	8.0	7.4	8.4	8.8	7.1	5.6	5.7	5.0	4.8	6.0	7.2	7.1	6.8

III. MONTHLY YIELD OF SOUHEGAN RIVER AT WILTON

Data: 1. Same as for I.

2. Average monthly evaporation at Chestnut Hill, Table 29, Chap. II.

3. Run-off data of Souhegan River at Merrimack, 1910-1923, U. S. Geological Survey, *Water Supply Papers*.

4. Topographic quadrangle (U. S. Geological Survey) sheets, Peterboro, Fitchburg, and Milford.

5. Souhegan River total drainage area at Merrimack 168 square miles. Effective water area 1.7 sq. miles. Reservoir site at Wilton, full-reservoir level at an elevation of 560.

Problem.

1. Using the topographic sheet assigned, sketch and ink in red the water-shed lines for the drainage-area boundary of Souhegan River considering as distinct portions and measuring with planimeter (a) the area above Merrimack and (b) the area above the reservoir site near Wilton.

2. Using the topographic sheet, determine separately the lake area and the swamp area for each of the drainage areas given in (1): (a) for the case with the reservoir, and (b) for the present conditions.

3. Correct and tabulate the monthly data of run-off in second-feet per square mile at the Wilton reservoir site basing the computations on the run-off at Merrimack, for the period, October, 1909,–September, 1923, and assuming an effective water area of 3.7 per cent above Wilton reservoir site. Drainage area = 65 sq. miles.

IV. FLOW-DURATION CURVES SOUHEGAN RIVER AT MERRIMACK

Data: From III, run-off at Merrimack.

Problem.

1. Using the data of monthly run-off in second-feet per square mile for the period October, 1909,–September, 1924, inclusive, prepare a flow-duration table on the calendar-year basis. Compute the mean for this 14-year period, of the driest month, next driest, etc., and the year, which should check with the mean for this period with months in calendar order.

2. Prepare a flow-duration table for the same period but on the total-period basis.

In tabulating, place the 14 lowest values in one column, the next 14 in another column, etc., determining the mean of each of these columns and checking as in (1).

3. Plot the results of (1) and (2), using months of the average year as abscissas and plotting each month as a line rather than a point, scale 1 in. = 2 months; and run-off in second-feet per square mile as ordinates, scale 1 in. = 1 sec.-ft. per square mile; lowest value at right of plot.

Also show by auxiliary scale at the right (a) discharge in second-feet for 168 sq. miles at Merrimack and (b) available horse power on wheel shaft (80 per cent efficiency) for the head of 56 ft.

Also show an auxiliary scale of abscissas as per cent of time, noting each 10 per cent with 0 per cent at the left.

V. FLOOD FLOW OF SOUHEGAN RIVER

Data: Appended.

Problem.

1. Determine a suitable value of C in Fuller's formula for average yearly flood, applicable to the Souhegan River, based on data of maximum daily discharge at Merrimack. Using this value of C , determine the average yearly flood for the Wilton reservoir site on the Souhegan River. Drainage area = 65 sq. miles.

2. Determine from the results of (1) the maximum crest flood to be expected on the Souhegan River at Merrimack for periods of 50, 100, and 1000 years, based on Fuller's method. Consider that there is no storage at Wilton or elsewhere.

3. Determine the maximum elevation to which water would rise in the proposed Wilton reservoir during a 1000-year flood, and the maximum flow over the spillway, assuming that the water level in the reservoir at the beginning of the flood day is at an elevation of 560 and that the distribution of flow into the reservoir during the middle 20 hr. of the flood day is parabolic, the flow for the first 2 and last 2 hr. of the day being constant. Use "mass-curve" method, page 184.

MAXIMUM DAILY DISCHARGE OF SOUHEGAN RIVER AT MERRIMACK, N. H., 1909-1923

Year	Discharge, second-feet	Year	Discharge, second-feet
1909-1910	4,350	1916-1917	2,570
1910-1911	2,670	1917-1918	1,500
1911-1912	2,330	1918-1919	2,490
1912-1913	2,890	1919-1920	2,970
1913-1914	2,810	1920-1921	3,570
1914-1915	3,850	1921-1922	3,050
1915-1916	2,970	1922-1923	2,970

TABLE OF AREAS FOR WILTON RESERVOIR SITE

Elevation	Area, Square Miles
460.....	0.16
480.....	0.38
500.....	0.57
520.....	0.81
540.....	1.09
560.....	1.45
580.....	1.83

SPILLWAY COEFFICIENTS¹

Head on Spillway, Feet	Coefficient <i>C</i> in $Q = CbH^{3/2}$
0.1.....	2.75
0.3.....	3.00
0.5.....	3.15
1.0.....	3.45
1.5.....	3.64
2.0.....	3.75
2.5.....	3.83
3.0.....	3.87
3.5.....	3.88
4.0.....	3.88
Spillway length.....	100 ft. at an elevation of 560.

¹HORTON, R. E., "Weir Experiments and Coefficients," U. S. Geol. Survey, *Water Supply Paper* 200, series 34, Plate XXV.

VI. PLANT CAPACITY AND OUTPUT—SOUHEGAN RIVER AT MERRIMACK

Data: Flow-duration-curve data from IV.

Load curve as in Fig. 44, Chap. III, with 50 per cent load factor.

Wheel efficiency 85 per cent.

Generator efficiency 94 per cent.

Assume pondage available at plant but no storage.

Problem.

1. Using the total-period flow-duration-curve tables, determine the horse power of wheel capacity requisite to utilize the flow available 4 months of the average year for each duration curve. Assuming 100 per cent load factor and no waste of water, determine the average annual output at switchboard for each curve of (a) primary power, (b) secondary power, and (c) total power.

2. If the plant in (1) runs on a 50 per cent monthly load factor, what will be the kilowatt-hours output of (a) primary power, (b) secondary power, and (c) total power, assuming that 10 per cent of the secondary power is lost due to waste, lack of pondage, etc. Use duration curve drawn on total-period basis.

3. If the pond at the dam has an area of 20 acres, (a) what daily draft of pond in feet will be required on days when only primary power is available, using 100 per cent utilization; (b) what pondage will be required expressed as a proportion of the average daily flow?

VII. HYDROGRAPH STUDIES OF STORAGE EFFECT—SOUHEGAN RIVER

Data: Computed monthly discharge for the period March, 1910,–February, 1916, inclusive, from controlled area above Wilton reservoir site and from uncontrolled area below reservoir site, compiled from III.

Assume storage and power development as follows:

1. Wilton reservoir site, storage capacity 2100 mill. cu. ft., spillway level at an elevation of 560.

2. Power development at Merrimack, net head 56 ft.

Problem.

1. Plot on sheet of profile paper (Plate A, 4 by 20, 24 in. long), scales to be approved by instructor, hydrographs of (a) uncontrolled area at bottom of sheet; (b) controlled area in middle of sheet, and leave room for reservoir-depletion curve at top of sheet.

2. Ascertain and show by means of hydrographs and table what best amount of flow can be obtained at Merrimack for each month of the 7-year period, March, 1910–February, 1917, by the two methods, *viz.*:

a. Storage at Wilton to be used yearly, as far as practicable.

b. Storage at Wilton to be used to give the best primary flow.

Show the modified flow at Wilton and reservoir-depletion curve for each of the two cases.

3. Construct on 8½- by 11-in. cross-section paper a flow-duration curve (total-period basis) of the modified flow at Merrimack, with yearly use of storage, using scales of second-feet at left and second-feet per square mile at right. Show also curve without storage effect.

Determine from these curves the proportion of reservoir capacity used during the average year.

4. What wheel capacity is necessary to utilize the modified flow (yearly use of storage) available one-third of the time? With 100 per cent utilization factor, what will be the yearly output in kilowatt-hours at switchboard of (a) primary power, (b) secondary power, (c) total power? Use wheel efficiency 85 per cent; generator efficiency 94 per cent.

VIII. SOUHEGAN RIVER—ATHERTON FALLS POWER DEVELOPMENT MAIN DAM

Data: Topographic plan of dam and power site (Fig. 310). At dam site, assume ledge rock suitable for foundations at an elevation of 135 in river bed; at 2 ft. below surface on each bank; and varying from 2 ft. at shore to 8 ft. at an elevation of 180 on west bank.

Spillway of dam to be concrete with crest at an elevation of 165, and top of flashboards at an elevation of 170. Spillway length to be 200 ft.

Remainder of dam on east bank to be concrete abutment section; on west bank, of earth with concrete core wall.

Design of Spillway Section.—Spillway section to be stable under a head of water of 8 ft. on the crest with no assumed upward water pressure. Assume tentative section

of maximum height, and investigate for line of resistance, sliding, etc., modifying section as necessary. Line of resistance should lie within middle third, and the angle with the vertical of resultant pressure on any joint should not exceed $\tan^{-1} 0.55$. Show conditions for each 10-ft. joint level of final section.

Also, for the section as determined, show line of resistance and direction of resultant joint pressures, assuming one-third upward water pressure over the joint area.

Abutment section on east bank, top at an elevation of 175, need not be designed in detail but may be based on dimensions of spillway section.

Design of Earth Section.—Adopt suitable top width and upstream and downstream slopes for given conditions. Concrete core wall is to be of suitable proportions and to extend into ledge rock. Design also a concrete retaining wall between earth and spillway sections.

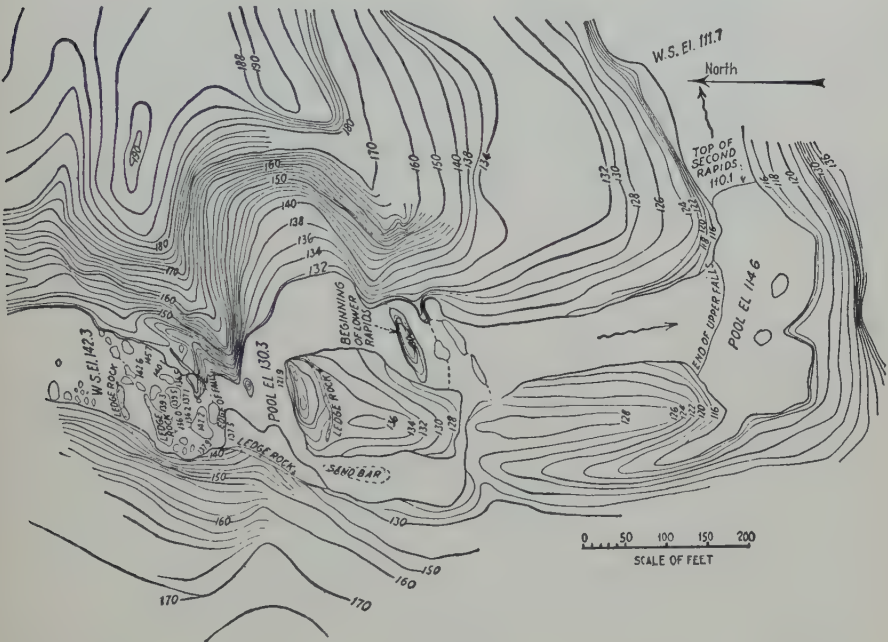


FIG. 310.

Appurtenances.—Prepare details for flashboards, pins, etc., specifying kind and full dimensions. Boards are to be unplanned spruce and are to go out when the head of water on their top exceeds about 2 ft.

Drawings.

a. On tracing paper (size 24 by 30 in., outside) make completed pencil drawings of spillway section, with lines of resistance, force polygon, etc. Show construction detail of spillway, abutment section, and retaining wall, with all dimensions in feet and inches to nearest $\frac{1}{8}$ in., elevations to nearest 0.01 ft.; also flashboard details with all construction dimensions and maximum section through earth portion of dam.

b. Show in profile the location and elevation of spillway, abutment, retaining wall, earth portion, and core wall. On plan print of site, show these in plan.

Title for (a) (in lower right-hand corner of sheet):

SOUHEGAN RIVER
ATHERTON FALLS POWER DEVELOPMENT
MAIN DAM
Scales as Noted

Date

Name

Also appropriate subtitles neatly and plainly lettered.

Drawings are to be in pencil in completed form. Bind drawings in (b) with report. Submit (a) separately.

IX. SOUHEGAN RIVER—ATHERTON FALLS DEVELOPMENT—PENSTOCK

Data:

Plant capacity 1600 hp. on wheel shaft.

Capacity factor 60 per cent; turbine efficiency 85 per cent.

Effective head 56 ft. $\pm$.

Penstock location on left bank following as direct a route as possible from dam (VIII) to a power house with center line of vertical wheel units at about an elevation of 120, located below bend in river. Penstock to be of reinforced concrete but with designs and cost estimates also for wood-stave and steel pipe. Center-line pipe at dam at about an elevation of 150.

Topographic plan of penstock location (see Fig. 310).

Materials and Stresses.—Allow one-third increase in pressure for water hammer. For friction factors, use mean values in Table 67, page 206, King's "Handbook of Hydraulics," 1929, formula

$$h_f = k_1 \frac{1}{d^{1.25}} \frac{v^2}{2g}$$

Reinforced Concrete—Steel reinforcement bars, square twisted, $f_t = 16,000$ lb. per square inch. Use one size for hoops and one size for longitudinal bars if practicable.

Wood Stave.—Steel bands $\frac{1}{2}$ - or $\frac{3}{4}$ -in. rounds; $f_t = 12,000$ lb. per square inch; spacing limits 3 to 8 in.

Staves, No. 1 redwood, $2\frac{1}{2}$ in. thick, dressed; f_b of bands on staves = 650 lb. per square inch.

Steel.—Minimum plate thickness $\frac{1}{4}$ in.; vary plate thickness by sixteenths; f_t on gross section of plate = 10,000 lb. per square inch.

Cradles.—Space 10 ft. on centers for wood-stave pipe; space to give bearing load on soil of $2\frac{1}{2}$ tons per square foot for steel pipe.

Design for cradles with schedule of quantities will be furnished.

Problem.

1. Plot curves of annual cost of pipe and power lost (scales: 1 in. = 2 ft. in diameter; 1 in. = \$0.50) for concrete pipe and determine best size.

2. Design pipe for best size with center line at ground level and 2 ft. of backfill over top of pipe.

3. Design wood-stave pipe of same capacity with invert of pipe at ground level, supported by reinforced-concrete cradles.

4. Design steel pipe, same conditions as in (3).

Drawings, in pencil (scales as directed).

1. Profile of penstock line showing ground surface, pipe, and maximum and minimum hydraulic grades for concrete pipe (scale: 1 in. = 20 ft.).

2. Plan of penstock line on accompanying print sheet (Fig. 310). Use radius of curvature of 100 ft. for concrete pipe line, except at dam where a 25-ft. radius is allowable.

3. Detail cross section (scale: $\frac{1}{2}$ in. = 1 ft.) of each of concrete, wood-stave, and steel penstock with steel schedules.

Estimates.—Give itemized quantities and cost estimates, total and per linear foot, of (1) concrete, (2) wood stave, and (3) steel penstock.

Cost Data.

Value of 1 hp. year undeveloped.....	\$5, \$10, and \$15
Annual fixed charges on pipe.....	10 per cent
Concrete for pipe in place.....	\$20 per cubic yard
Reinforcing bars in place.....	6 cts. per pound
Excavation or fill.....	\$1.00 per cubic yard
Wood stave in place.....	\$100 per M ft. b.m.
Steel bands, etc., in place.....	5 cts. per pound
Steel plate in place.....	6 cts. per pound
Cradles, design as given.....	\$20 each

X. SOUHEGAN RIVER—ATHERTON FALLS DEVELOPMENT—POWER HOUSE STUDY

Data: Assume two vertical units with single-floor type of station and vertical conical draft tube. Transformers and high-tension connections in outdoor yard.

Wheel units:

S. Morgan Smith type F-1, 47 in., 750 hp., 225 r.p.m., head 56 ft.

Dimensions as follows (see Fig. 164, Chap. VII); A, 54 in.; B, 60 in.; C, 103 in.; D, 7½ in.; I, 24 in.; J, 120 in.; K, 120 in.; L, 84 in.

Generator units:

General Electric Company, vertical 500 kw., 225 r.p.m., three phase, 60 cycles, 2300 volts.

Dimensions as shown on Fig. 311.

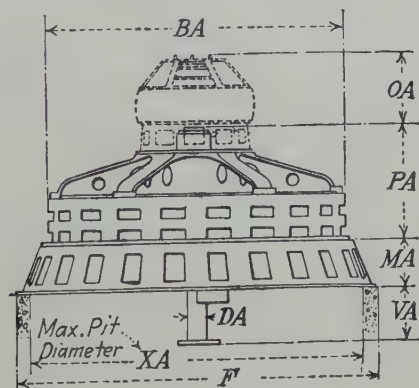


FIG. 311.—Vertical water-wheel-driven generator.

Type and Class.

Type ATB—32-pole—625-kva. (500-kw., 0.8 power factor)—225-r.p.m.—2300-volt—3-phase, 60-cycle vertical water-wheel-driven generator.

BA	DA	F	MA	PA	OA	VA	XA
88	6	107.5	14¼	37½	23¾	17	102

Problem.

Make sketches in pencil, ready to trace, of power-house layout on scale of 1 in. = 10 ft. with general dimensions.

1. Show plan of power house, with arrangement of units and accessories and penstock connections.

2. Show cross section of power house through a wheel unit.

XI. SOUHEGAN RIVER—ATHERTON FALLS DEVELOPMENT—COST ESTIMATES

Data: Plans of dam, penstock, and power house from VIII, IX, and X.

Penstock cost estimates from IX.

Cost data as follows:

Wheel units:

Wheel, f.o.b. York.....	\$9000	
Governor (Woodward).....	1600	
Butterfly valve.....	1600	\$12,200

Freight, 4000-lb. wheel (York, Pa.) at $32\frac{1}{2}$ cts..... \$130

2400-lb. governor (Rockford, Ill.) at 1.45 cts.. 35

\$165, say, 170

Erection, 10 per cent of \$12,200..... 1,220

Total cost erected..... \$13,590

Say, \$13,600

$$\frac{\$13,600}{750} = \$18.20 \text{ per horse power}$$

Generator (similar to that used):

General Electric Company, 350 kw., 2300 volt, three phase, 60 cycle, 200 r.p.m.

Cost, f.o.b. works..... \$6000

Freight, say..... 150

Erection, say..... 500

Total cost erected..... \$6650, say, \$6700

$$\frac{\$6700}{350} = \$19.20 \text{ per kilowatt}$$

Generator used, 500 kw., 225 r.p.m., at say \$19 per kilowatt... \$9500 erected.

Switchboard, wiring, transformers, etc.: Assume at \$6 per kilowatt

Crane: Assume at \$1500.

Unit prices for dam:

Rock excavation..... \$5 per cubic yard

Earth excavation..... \$1 per cubic yard

Concrete..... \$16 per cubic yard

Handling water..... \$1000 lump sum

Assume that preparation of rock foundation, cut-offs, etc., will require rock excavation to a depth of 2 ft. over foundation area.

Power house: Cost estimate to be based on data in Table 104, Chap. XII.

Engineering and contingencies: Assume at 15 per cent of construction cost.

Land: Assume at \$2000.

Interest during construction: Assume at 6 per cent of construction cost.

Problem.

1. Make detailed cost estimates for dam, with summary of quantities and costs.
2. Make approximate cost estimate for power house and equipment.
3. Make itemized summary of cost of entire development, showing also cost per installed horse power.

XII. SOUHEGAN RIVER—ATHERTON FALLS DEVELOPMENT—COST AND VALUE OF POWER

Data: Output from VI; cost from XI.

Cost of operation, assume at 0.15 ct. per kilowatt-hour.

Auxiliary steam power at 1.5 cts. per kilowatt-hour (total cost).

Load factor 50 per cent, as in VI.

Primary-power sale price, 1.1 cts. per kilowatt-hour.

Secondary-power sale price, 0.3 ct. per kilowatt-hour.

Problem.

1. Estimate cost per kilowatt-hour of power for average year.
2. Estimate yearly gross and net income (*a*) without and (*b*) with steam auxiliary to make all primary power up to 50 per cent of wheel capacity.
3. Estimate value of water privilege based upon (*a*) and (*b*) in (2)

XIII.—PRELIMINARY DESIGN AND LAYOUT

(After completion of Chaps. I-X, inclusive)

Given the following conditions for a hydroelectric plant:

Development of concentrated fall at a concrete masonry dam.

Top of non-overflow section of dam at elevation 656.5.

Top of 8-ft. flashboards on spillway at elevation 650.

Minimum reservoir level at elevation 610.

River bed of ledge rock at elevation 478 at heel of dam and elevation 450 at a distance 235 ft. downstream, varying uniformly between these two points.

Normal tailrace water level at elevation 472; flood level at elevation 487.

Three vertical wheel units, each to develop about 50,000 hp. when water level is at top of flashboards.

Racks may be constructed vertically assuming a mechanical raking device.

Waterway to be a steel-plate penstock for each unit, built through the non-overflow section of dam with power house at downstream toe of dam.

Generators to be three phase, 60 cycle, 6600 volts; for data on dimensions, see page 436.

Other features and details to be assumed consistent with general practice.

a. Make an approximate layout for this development, showing all necessary computations and assumptions used as a basis for design. Show a longitudinal section through the center of a unit from intake to tailrace on a scale of 1 in. = 40 ft. with elevations and dimensions, and additional notes or sketches as necessary to same scale or larger.

b. Determine characteristics of wheel and generator units, spacing of units, and approximate dimensions of power house.

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